SEISMIC DESIGN AND RETROFIT MANUAL FOR HIGHWAY BRIDGES

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FOREWORD

The Implementation Package provides practical guidelines for the seismic design and retrofit of highway bridges. The manual should be useful to both beginners and experts in seismic design. It emphasizes short and medium span bridges that are typical of current practice throughout the United States. The manual is appropriate for a wide range of common bridge types in all seismic zones across the country.

Copies of the manual are being distributed to FHWA Region and Division offices and to each State highway agency. Additional copies of the manual can be obtained from the National Technical Information Service, Springfield, Virginia 22161.

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NOTICE

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16. Abstract

This manual is a guide to the seismic design of bridges. It presents the basic principles of seismology, structural dynamics and structural form as they relate to highway bridge structures. Seismic design concepts are presented which highlight the importance of simplicity, symmetry and integrity for bridges. Examples of acceptable structural form and form to be avoided are included.

Past earthquake damage is reviewed and a philosophy for seismic bridge design is presented. Analysis methods for calculating design forces and displacements are developed, and relevant computer software is discussed. Numerical examples are used to illustrate design concepts and the recommended design procedures.

A methodology for seismic retrofitting is also presented, which includes discussion of bridge evaluation procedures, capacity/demand ratios and design concepts for strengthening and upgrading existing vulnerable bridges.

The philosophy and procedures presented in this manual are consistent with those recently developed by the Applied Technology Council (ATC) with funding from the Federal Highway Administration (FHWA) and now adopted by the American Association of State Highway and Transportation Officials (AASHTO).

This manual synthesizes the criteria and recommendations of the above agencies (ATC, FHWA, and AASHTO) and incorporates recent work by the California Department of Transportation and the Earth Technology Corporation of California.

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CONVERSION FACTORS TO SI METRIC UNITS

Multiply	by	to obtain
inches (in)	0.0254	meters (m)
inches (in)	2.54	centimeters (cm)
inches (in)	25.4	millimeters (mm)
feet (ft)	0.3048	meters (m)
yards (yd)	0.9144	meters (m)
miles (mi)	1.609	kilometers (km)
degrees (°)	0.01745	radians (rad)
acres (acre)	0.4047	hectares (ha)
acre-feet (acre-ft)	1233.	cubic meters (m^3)
gallons (gal)	3.785×10^{-3}	cubic meters (m^3)
gallons (gal)	3.785	liters (l)
pounds (lb)	0.4536	kilograms (kg)
tons (2000 lb)	907.2	kilograms (kg)
pounds force (lbf)	4.448	newtons (N)
pounds per sq in (psi)	6895.	newtons per sq m (N/m^2)
pounds per sq ft (psf)	47.88	newtons per sq m (N/m^2)
foot-pounds (ft-lb)	1.356	joules (J)
horsepowers (hp)	746.	watt (W)
British thermal units (Btu)	1055.	joules (J)

Some Definitions

newton - force that will accelerate a 1 kg mass at 1 m/s^2
joule - work done by a force of 1 N moving through a displacement of 1 m
1 newton per sq m (N/m^2) = 1 pascal (Pa)
1 kilogram force (kgf) = 9.807 N
1 gravity acceleration (g) = 9.807 m/s^2
1 hectare (ha) = 10,000 m^2
1 kip (k) = 1000 lb = 4448 N = 453.6 kgf = 0.5 ton

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LIST OF SYMBOLS

A	acceleration coefficient
A	wave amplitude
A_g	maximum ground acceleration
A_r	bonded area of elastomer
A'_s, A'_w	effective shear areas
a	acceleration
a_g	ground acceleration
a_x, a_y, a_θ	components of acceleration
B	buoyancy load
C, c_s	seismic coefficient, lateral load coefficient
c	damping coefficient
D	dead load
D	maximum displacement, initial displacement
D	response magnification factor
D_g	maximum ground displacement
DR	dead load reaction
d	displacement
$d(t)$	displacement as a function of time
d_x, d_y	components of displacement
E	earth pressure
E, E_c, E_s	modulus of elasticity
EQF	earthquake load on foundations
EQM	earthquake load on structural members
e	exponential constant (2.7183)
F	force, maximum value of $f(t)$
F_D	damping force
F_I	inertia force
F_{Ix}, F_{Iy}	components of inertia force
F_S	spring force
F_v	shear force
F_{vd}	dependable shear strength
F_{vi}	ideal shear strength
F_{vn}	required shear strength
F_x, F_y	components of force

LIST OF SYMBOLS

F_o	maximum force
$f(t)$	force as a function of time
f	unit friction load on pile
f_{max}	maximum unit friction on pile
G	shear modulus of soil
G_r	shear modulus of elastomer
g	gravity acceleration
H, h	pier or abutment height
H	horizontal force
I_c, I_s, I_w	moments of inertia
K, k	stiffness
K_b	bearing stiffness
K_h	modulus of horizontal subgrade reaction
K_h, K_v	components of stiffness in horizontal and vertical directions
K_L, K_T	components of stiffness in longitudinal and transverse directions
K_p	pier stiffness
K_x, K_y, K_θ	components of stiffness in (x,y,θ) directions
K_o	stiffness coefficient for circular footing
k	spring stiffness
k^*	effective stiffness
L	span length, total bridge length
L	column height
L_m	pile length to fixity for equivalent moment
L_s	pile length to fixity for equivalent stiffness
M	Richter magnitude for earthquakes
M,m	mass
M_L, M_T, M_θ	components of mass (translational and rotational mass inertia)
M_d	dependable flexural strength
M_i	ideal flexural strength
M_n	required flexural strength
M_o	overstrength flexural capacity
M_{max}	maximum bending moment
M_T, M_B	top and bottom flexural moments in a column

LIST OF SYMBOLS

N	number of blows per foot, standard penetration index
N	number of earthquakes of given magnitude
N	seat width
n	damping ratio
n	number of columns in a bent
n_h	constant of horizontal subgrade reaction
P, P_o	load
p	natural frequency
$p(t)$	load as a function of time
p_o	maximum value of $p(t)$
p_o, $p_e(x)$	load intensity (force per unit length)
Q	axial load on pile
Q	vertical seismic force
Q	torque
δQ	elemental torque
q	pile tip resistance
q_{max}	maximum tip resistance
R	focal distance
R	force reduction factor, response modification factor
R	radius of circular footing
R_0, R_1, R_2, R_3	equivalent radii for rectangular footings
r	frequency ratio
r_{ab}	capacity/demand ratio
S	site coefficient
S_a, S_v, S_d	spectral acceleration, velocity and displacement
SF	stream flow pressure
S_d	dependable strength
S_i	ideal strength
S_o	overstrength
T, T_p	period of vibration
T_{max}	maximum kinetic energy
T_r	total thickness of elastomer
t	time

LIST OF SYMBOLS

U	potential energy
V	shear force
v	velocity
$v(t)$	displacement as a function of time
$\ddot{v}_g$	ground acceleration
$\ddot{v}_{go}$	maximum ground acceleration
$v_s(x)$	displacement as a function of distance along span
W	width
W	weight
W_E	work done by external load
w	load per unit length
$w(x)$	weight per unit length
x,y	Cartesian coordinates
z	axial displacement of pile
z_c	critical axial displacement of pile
α	constant in beam stiffness formula
α	foundation shape correction factor for footings
α	area (integral) under lateral deflected shape of superstructure; is proportional to effective lateral stiffness
β	constant in cantilever stiffness formula
β	embedment factor for footings
β	area (integral) under weighted lateral deflected shape of superstructure
β	frequency ratio
γ	area (integral) under the square of the lateral deflected shape weighted by the weight per unit length of the superstructure; is proportional to effective mass
Δ	deflection
Δ_u, Δ_y	maximum and yield deflections of a single column pier.
θ	phase angle

LIST OF SYMBOLS

θ	rotation
μ	ductility factor = Δ_u / Δ_y
ν	Poisson's ratio for soil
ξ	damping ratio
π	3.141593
ρ	maximum displacement
Φ	strength reduction factor
Φ_o	overstrength factor
Φ_u, Φ_y	maximum and yield curvatures of member section
Φ'	effective soil internal angle of friction
ω	frequency, natural or forced
$\bar{\omega}$	frequency, forced
ω_D	damped natural frequency

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ATC	Applied Technology Council
Caltrans	California Department of Transportation
C/D	Capacity/Demand Ratio
FHWA	Federal Highway Administration
IC	Importance Classification
SPC	Seismic Performance Category

CHAPTER 1 INTRODUCTION

Earthquakes damage civil engineering structures every year and bridges are no exception. Historically, bridges have proven to be vulnerable to earthquakes, sustaining damage to substructures and foundations and in some cases being totally destroyed as superstructures collapse from their supporting elements (figure 1). In 1964 nearly every bridge along the partially completed Copper River Highway in Alaska was seriously damaged or destroyed. Seven years later, the San Fernando earthquake damaged more than sixty bridges on the Golden State Freeway in California. This 1971 earthquake is estimated to have cost the State approximately $100 million (1984 dollars) to repair and replace these bridges, including the indirect costs due to bridge closures. Both Japan and Chile have also experienced seismic damage to modern bridges in recent years.

The poor seismic performance of bridge structures is surprising in view of the substantial advances made in design and construction for vehicular (vertical) loads. For more than a century, bridge spans have been pushed further than before, alignment has become increasingly complex and aesthetic requirements have become more demanding. Nevertheless, these demands have been satisfied by use of innovative materials and, more recently, computer based analysis and design methods. However, similar advances have not been made for the seismic performance of bridges as evidenced by the Anchorage and San Fernando earthquakes.

The reason for this apparent paradox is that for live load, the critical element in a bridge is the superstructure whereas for seismic loads, the critical elements are the substructures and foundations and their connections to the superstructure. The advances in the state-of-the-art have been related to the superstructure with little or no attention being given to the substructures and their performance under high lateral load. Fortunately this situation has changed in the last ten years.

Following the defective performance of bridges in the San Fernando earthquake, the Federal Highway Administration (FHWA) and the California Department of Transportation (CALTRANS) began exhaustive studies into the seismic performance of bridges. This intense effort has resulted in a series of publications, interim specifications and seismic design guidelines for both new and existing bridges.

CALTRANS adopted new seismic design criteria in 1973, and the American Association of State Highway and Transportation Officials (AASHTO) published a modified version of these criteria as interim specifications in 1975. These have since been incorporated in subsequent editions of the Standard Specifications for Highway Bridges [reference 1] and are the basis of seismic criteria for bridges nationwide. CALTRANS has refined and updated its criteria continuously over the last decade [reference 2]. They are now more rigorous than the AASHTO Standard Specifications and reflect the higher seismic risk in the State of California. In addition the FHWA funded the Applied Technology Council (ATC) of California to prepare a synthesis report on seismic bridge

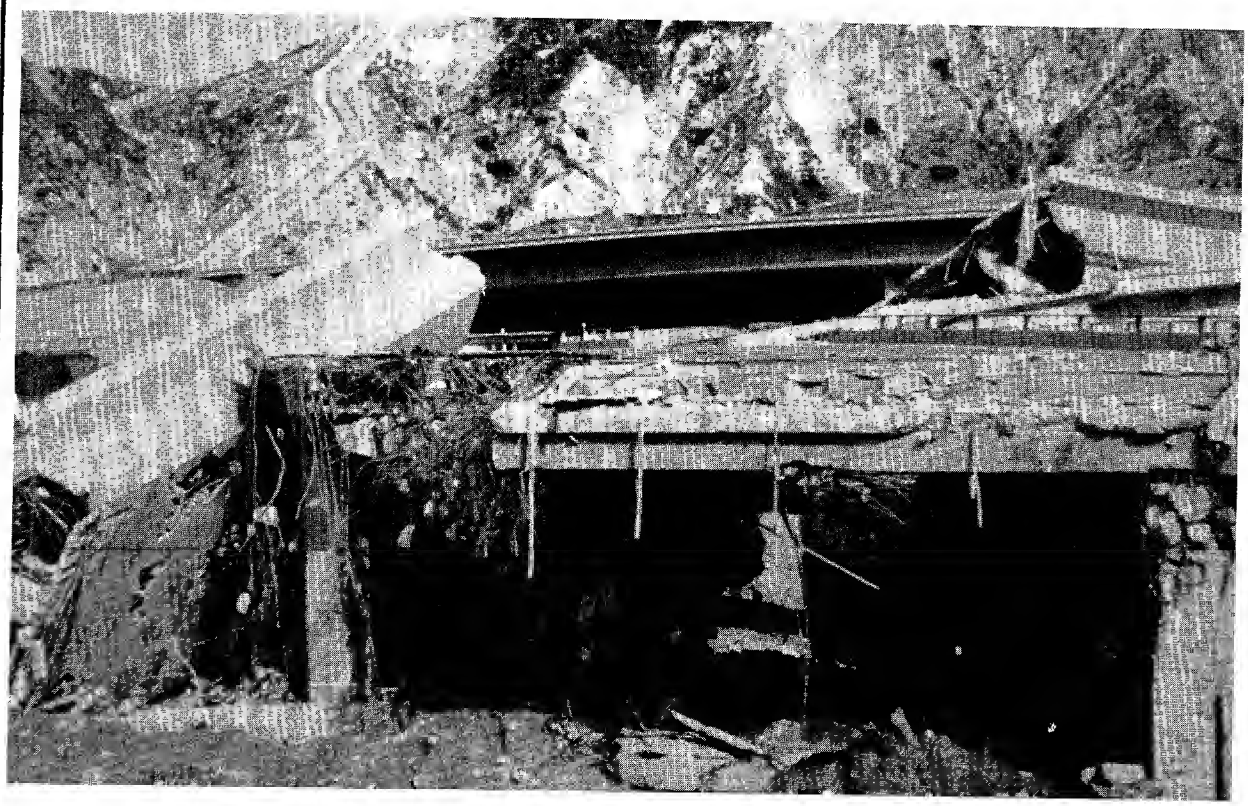


Figure 1: Damaged Overcrossing, Golden State Freeway and Foothill Freeway Interchange, San Fernando Earthquake, 1971

design based on the results of research recently undertaken within the United States and in several foreign countries (principally New Zealand and Japan). Published in 1981 as set of Seismic Design Guidelines for Highway Bridges [reference 3], this report, widely known as ATC-6, is a state-of-the-art document which includes the most recent developments and design practices for the seismic design of bridges. Written in a code format, with an extensive commentary, it was adopted by AASHTO in 1983 as an approved alternate Guide Specification for seismic design [reference 4] in all fifty states.

Following the successful completion of ATC-6, the FHWA subsequently funded two further projects in this area. The first of these was for the preparation of a companion set of guidelines for the seismic retrofit of existing bridges. These were published in 1983 by the Applied Technology Council under project ATC 6-2 [reference 5] and are unique in that they are thought to be the only set of retrofit guidelines in print. The second FHWA contract was for the preparation of a seismic design manual for bridge foundations. Recently completed by the Earth Technology Corporation, this publication is another state-of-the-art reference work for bridge designers [reference 6].

In view of this wealth of literature, it is not surprising that bridge designers feel overwhelmed by these rapid changes in design procedures and engineering practices. Aware of this situation, FHWA funded the Engineering Computer Corporation in 1980 to prepare a Workshop Manual on the Seismic Design of Bridges, and to follow through with a series of workshops to help bridge engineers assimilate and become familiar with these new design procedures. However, at that time, the ATC Guidelines and the Foundation Manual [references 4, 5 and 6] were not complete and a revision to the Workshop manual was considered necessary in the light of these subsequent publications.

This present manual is the result of this revision. However, rather than a "revision", this manual is a completely new document. It might be thought of as a beginner's guide to seismic bridge design but it is hoped that the expert will also find something of interest in these pages. The emphasis of this work is on short- and medium-span bridges that are typical of current design practice throughout the United States. As will be seen, the superstructure type (girder or truss) is less important than the continuity of the superstructure, its connection to the substructures, and the design of the substructures and foundations. Therefore, this manual is appropriate to a wide range of common bridge types in all seismic zones across the United States.

After a brief survey of relevant **seismology** in chapter 2, basic **bridge dynamics** are introduced in chapter 3. Some background is assumed in engineering mathematics at this point, but the explanations of dynamic behavior are deliberately non-mathematical in the hope of developing a "feel" for bridge response to dynamic loads. Even the notation is simplified to minimize the number of road-blocks to understanding the basics of bridge dynamics.

Chapter 4 explains the **design philosophy** currently adopted in the AASHTO Guide Specifications for seismic design and gives some historical background to this philosophy. Accordingly, the past performance of bridges in earthquakes is reviewed and typical damage and failure mechanisms are discussed. Structural form as it affects bridge seismic performance is discussed in detail in chapter 5 under the heading **design concepts**. Good form is illustrated and the importance of simplicity, symmetry, and integrity in a bridge is highlighted. Structural form to be avoided is also illustrated.

Design loads and their background are presented in chapter 6, especially in the context of the AASHTO Guide Specifications. Of particular note here is the introduction of explicit Response Modification Factors for reducing elastic forces to obtain design loads. Chapter 7 overviews the calculation of member forces and displacements, given the design loads. These **design forces and displacements** are found from recommended methods of analysis (simplified where possible into a single mode procedure) and used to proportion the columns, connections, footings and foundations of the bridge. Several **design examples** are given in chapter 8 to illustrate the procedures outlined in the earlier chapters.

Guidelines for the **retrofit of existing bridges** are summarized in chapter 9. It is an attempt to consolidate the main points of the ATC 6-2 report [reference 5] into one chapter to give an overview of the philosophy and some of the options available to the engineer engaged in retrofit. It is not possible to condense all the relevant facts into one chapter and the designer needing greater detail is referred to the source document.

Chapter 10 presents some **comparative analyses** performed with the computer program SEISAB. These analyses illustrate the usefulness of SEISAB and the limitations of single-mode modelling for bridge seismic response.

CHAPTER 2 BASIC SEISMOLOGY

The level of seismic force that a bridge will be subjected to depends on the seismicity of the region where the structure is to be built. Seismology is the science of earthquakes and related phenomena and it is through this science that seismic activity and thus the seismic design loads for a bridge may be quantified.

In this chapter some basic concepts of seismology are introduced. Definitions of terms commonly used in seismology are provided in a glossary at the back of this manual. Explanations of the more important concepts are, however, given in this chapter. The source of earthquake activity is then discussed and the nature of the motions during an earthquake described. Features of these motions which are especially pertinent to a bridge designer are emphasized, in particular the prevalent directions, frequency content of the motions and the possibility of long period waves which may influence the design of large bridge structures.

In addition to the effects of earthquakes, probability information concerning their magnitude and occurrence is required to develop design forces. The consideration of motions from all possible events plus the probabilities of their occurrences is then the basis for determining the design earthquake for a particular site. The design earthquake will also depend on local conditions (geology and soil profile) at the proposed site. The importance of these items is discussed.

This chapter is not intended to provide an in-depth knowledge of seismology and in fact such detailed knowledge is not generally required of the bridge designer. The end product of the magnitude and risk studies is generally available in code form as a seismic design coefficient or a design spectrum. However, a basic understanding of the mechanisms and effects of earthquakes will be helpful in assessing whether code specifications are sufficient for a particular site and in deciding how to account for unusual conditions which may exist at a site. When necessary, further information can be found in references 7, 8, 9, 10 and 11.

2.1 TERMS USED IN SEISMOLOGY

Several terms commonly used in seismology are explained in a glossary appended to this manual. Amplification of some of the more important terms is given below.

Intensity

The intensity of an earthquake is a subjective measure of the effects of an earthquake at a given location. It refers to the level of shaking at a specified place and therefore a single earthquake will have a series of intensities, depending on where it is measured. Various intensity scales have been proposed to date, with perhaps the two most popular

being those of Rossi and Forel (Europe, 1880's) and Mercalli (Italy, 1902). The former scale (with ten numerical divisions) failed to distinguish between "strong" events over a certain intensity. The later scale in modified form is widely used today. It has twelve numerical divisions, and is listed in detail in appendix A.

Curves drawn on a map which pass through areas of equal observed intensity are called isoseismals. They are usually used to define the boundaries between regions with successive intensity ratings. A typical isoseismal map is shown in figure 2.

Magnitude

Magnitude is an instrumental measure of the size of an earthquake, independent of the site of observation. The measurement is based on the principle that the amplitudes of ground motions produced by earthquakes are a measure of the energy released by the earthquake. Although this is difficult to quantify in practice, it is the basis of the most commonly used magnitude scale, the Richter magnitude, denoted by M. Richter defined the magnitude of a local earthquake as the logarithm to the base 10 of the maximum seismic wave amplitude (in thousandths of a millimeter) recorded on a standard seismograph at a distance of 100 kilometers from the earthquake epicenter.

$$\text{i.e. } M = \log_{10} A \tag{1}$$

where A is the wave amplitude expressed in microns.

For instruments not located 100 km from the epicenter, a correction for distance is applied. Other corrections may also be applied to account for differences in instrument properties, and the type of seismic wave used to determine A. Despite these corrections, variations in the estimation of magnitude for the same earthquake are still possible. The Richter magnitude most frequently used by the media, when reporting a major earthquake, is based on surface wave amplitudes measured a thousand or more kilometers from the source.

Because of the logarithmic nature of the Richter scale, a unit change in magnitude corresponds to a tenfold change in wave amplitude (ground movement). Since the energy released in an earthquake is also logarithmically related to magnitude, a unit change in magnitude corresponds to a thirtyfold change in released energy. This fact can be illustrated by the observation that, in general, bridge structures are not usually damaged in earthquakes less than 5.5 in magnitude. However, a magnitude 6.5 earthquake can be devastating as demonstrated by the 1971 San Fernando earthquake (which was a 6.5 event) in which 66 highway bridges were damaged or destroyed. It is worth noting that it is now unlikely that a magnitude 6.5 event would damage new bridges to the same extent. This is because seismic design procedures for California bridges were substantially improved following this 1971 earthquake and bridges constructed since the mid-70's are expected to perform significantly better than pre-1971 structures.

Hypocenter, Epicenter and Focus

The hypocenter is that underground point where the initial rupture of rock occurs during an earthquake. The epicenter is a point on the earth's surface vertically above the hypocenter. Focus is a synonym for hypocenter. It is important to recognize that the epicenter and hypocenter do not indicate the center of energy release.

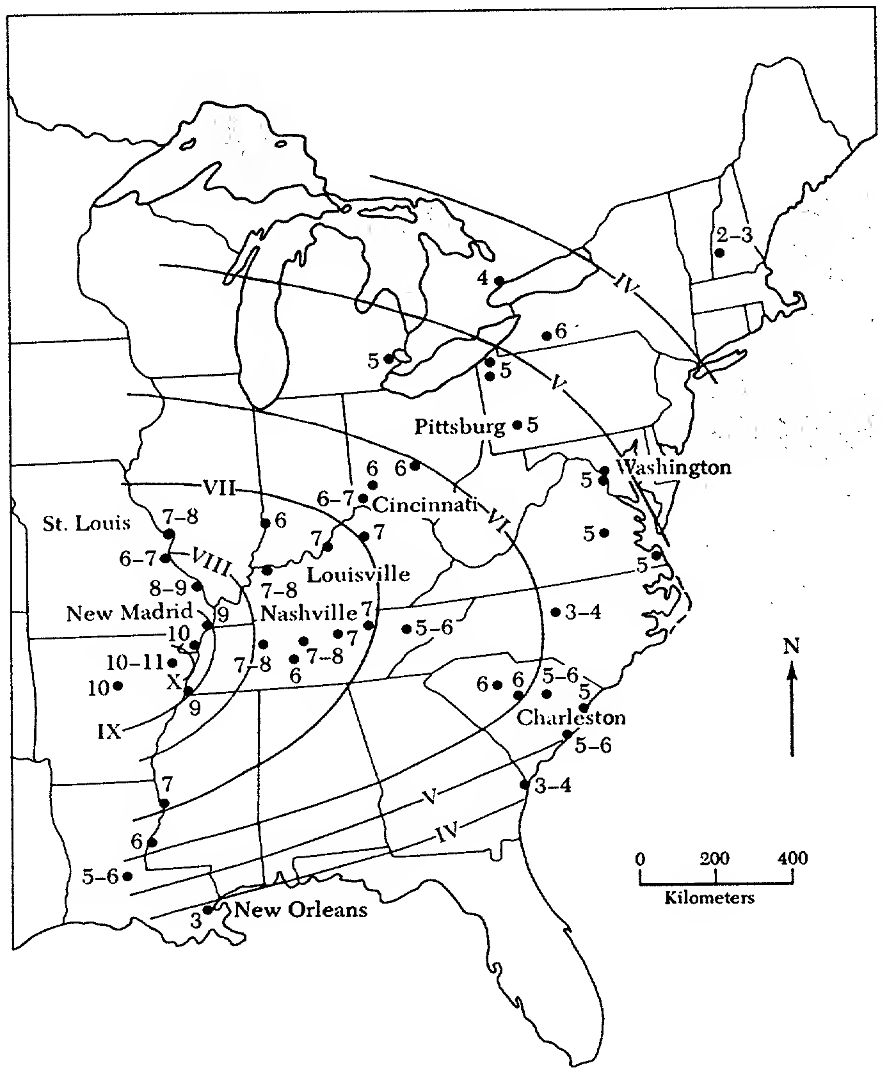


Figure 2: Isoseismals for the 1811 New Madrid Missouri Earthquake Based on the Modified Mercalli Scale (from Reference 7)

Faulting

Faulting refers to planes of weakness in the earth's crust, generally accompanied by relative movement of material on either side of the fault. Faults may be active (at least one movement within the last 35,000 years or two within the last 500,000 years) or inactive.

The fault cuts the ground (horizontal) plane along a line whose direction from north is called the **strike** of the fault. The fault plane itself may not necessarily be vertical and the inclination from the horizontal is called the **dip** of the fault (measured perpendicular to the strike). Three basic types of faults can thus be identified, based on the nature of the relative movement between material on either side of the fault plane.

When the rock on that side of the fault hanging over the fracture, slips downward relative to the other side, the fault is said to be **normal**. Conversely, when the overhanging side moves upwards the fault is called a **reverse** fault. Both normal and reverse faults produce vertical displacements seen at the surface as fault scarps, and are called **dip-slip** faults. In contrast, faults producing horizontal displacements are called **strike-slip** faults. If the motion of the far side of the fault is from right to left the faulting is **left lateral**; if it is from left to right the faulting is **right lateral**. Most faults produce a combination of horizontal and vertical motion, and are called **oblique** faults. These fault types are illustrated in figure 3.

Body and Surface Waves

When rupture along a fault occurs, the sudden release of energy sets off vibrations in the earth's crust. These vibrations can travel both within the earth's material (body waves) and on the earth's surface (surface waves). Figure 4 gives a visual interpretation of the various types of waves.

There are two major types of **body** waves - longitudinal or P waves and transverse or S waves. The **longitudinal waves** travel by compressions and dilations in the direction of propagation, and have the fastest speeds. They are denoted P for primary waves and travel at speeds of several miles per second. These waves can travel through both solid and liquid material. The **transverse waves** travel by shear distortions normal to the direction of propagation. Although they are denoted S for secondary waves, they transmit more energy than the P waves. S waves may also be plane-polarized. Those that cause motion in a vertical plane containing the direction of propagation are called SV waves (these are illustrated in figure 4); horizontally polarized waves are called SH waves. S waves cannot travel through liquids.

Surface waves are so called because their motion is restricted to close to the ground surface. As the depth below the ground surface increases, the wave amplitudes become less and less. There are two types of surface waves during earthquakes. The first is called a **Love** wave, whose motion is similar to that of an S wave horizontally polarized, except that its effects die out as depth increases. The second is called a **Rayleigh** wave, similar to a rolling ocean wave. Material disturbed by a Rayleigh wave moves in an elliptical path in the vertical plane containing the direction of propagation. Surface waves travel more slowly than body waves, with Love waves being generally faster than Rayleigh waves.

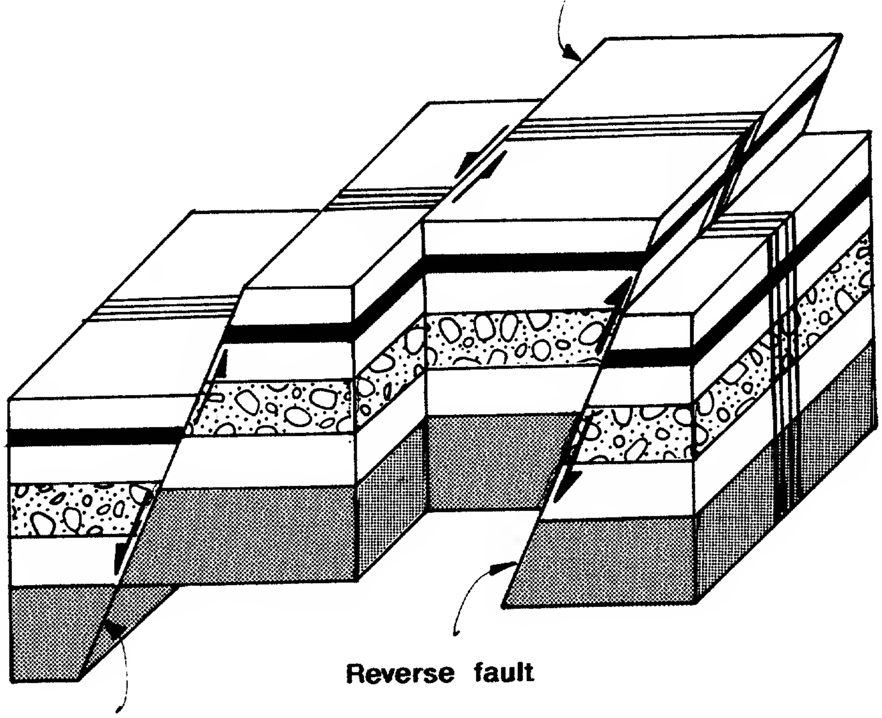
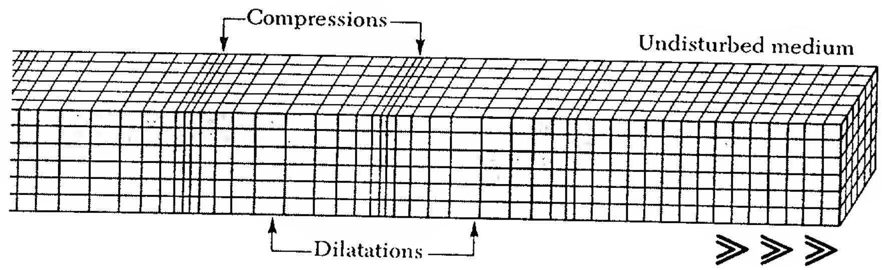
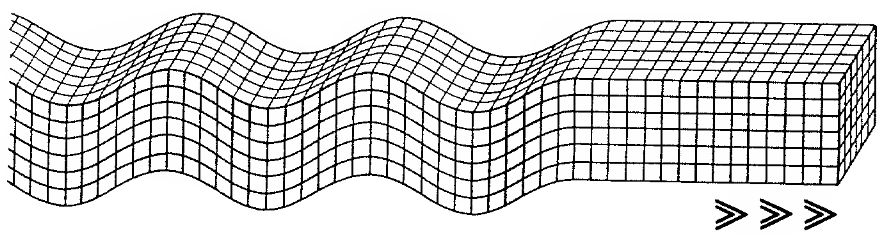


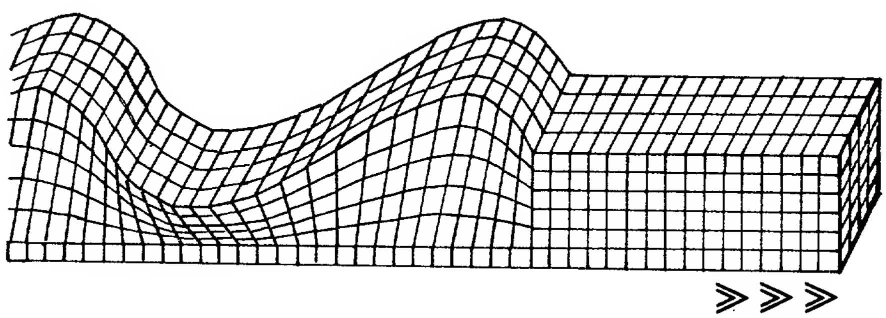
Figure 3: Definition of Fault Types
(after Reference 7)



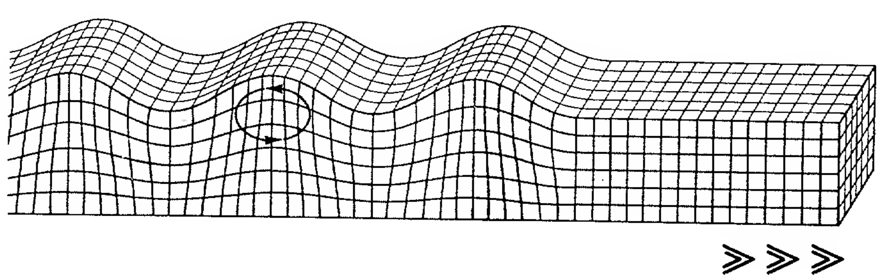
(a) P wave



(b) SV wave (SH wave similar but motion in horizontal plane)



(c) Love wave



(d) Rayleigh wave

Figure 4: Ground Motion due to Various Earthquake Waves (after Reference 7)

The amplitude of surface waves is generally insignificant compared to that of P and S waves at distances as much as five times the depth of the focus. However, at greater epicentral distances, the Rayleigh waves become very prominent.

2.2 EARTHQUAKE ACTIVITY

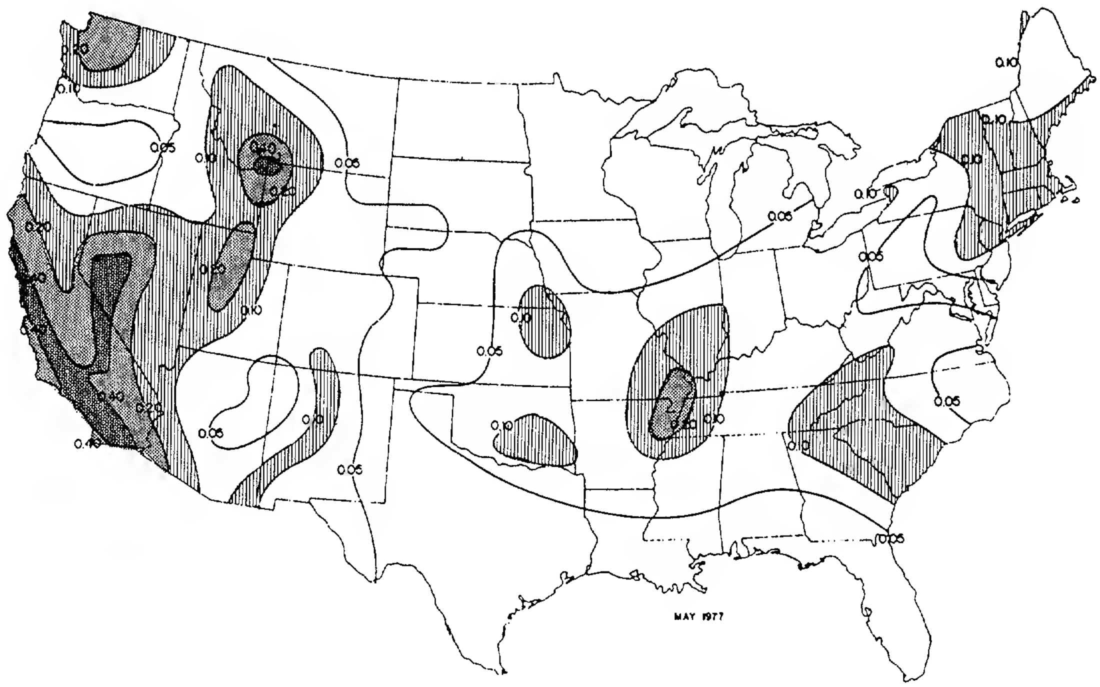
If the locations of the epicenters of all earthquakes are plotted on a map of the world over a period of a decade or two, the resulting pattern will not be random. Clear "belts" of seismic activity will separate large areas where almost no earthquake centers are located. Some of these belts are located along the edges of continents, while others occur in the middle of major oceans. Still other belts coincide with arcs of small islands, such as those in the Pacific, and the Aleutian chain swinging westward from Alaska. These belts of earthquake activity are dramatic evidence that the earth is not a solid, immoveable body. Rather, the theory used to explain this occurrence (backed up by clear geological evidence) is that the earth's crust consists of a number of large plates, slowly but surely grinding past each other. It is this relative movement at the plate boundaries that is the major cause of earthquakes. However, there are known occurrences of catastrophic earthquakes in regions away from plate boundaries. In the United States, the New Madrid earthquakes (1811-1812) in Missouri and the Charleston, South Carolina (1886) earthquake are dramatic examples of exceptions to the general trend.

The plates are typically 30 to 60 miles thick, and in fact, most of the damaging earthquakes have focal depths well within this range. In California, for example, all the known earthquakes to date have been shallow-focus, and in central California, almost all events have focal depths less than 10 miles.

The constant "grinding" of the edge of one tectonic plate past its neighbor is not in itself sufficient to explain the sudden release of energy during an earthquake. The theory coupling the relative movement between plates with the sporadic release of large amounts of energy in the form of earthquakes is known as the **elastic rebound** theory. The theory holds that an initially straight line perpendicular to the strike of the fault when the material is in an unstrained state, slowly warps as the plates move relative to each other. This warping causes a gradual buildup of strains in the material in the fault zone. This straining cannot continue indefinitely, for eventually the weakest rocks or those in the area of greatest strain will fracture. This fracture will be followed by an elastic rebound of strained material on either side of the fault, to its unstrained state. It is this rupture and subsequent rebound which is the immediate cause of earthquakes. On normal or reverse faults, this straining occurs in the vertical direction. After the earthquake, the two sides of the fault lock up again and further straining occurs.

The majority of earthquakes are not accompanied by visible fault displacements, and the rupture and rebound occurs below the earth's surface. When the earth's surface does rupture, relative displacements up to 20 feet can occur.

Seismically active regions in a global sense are apparent from the plot of earthquake epicenters. For the United States, a recent seismic risk map has been developed based on the distribution of these regions. This map (figure 5) gives the expected intensity of ground shaking in terms of **effective peak acceleration**. This can be thought of as



Contours indicate effective peak acceleration (expressed as a decimal fraction of gravity) that might be expected (with odds of 1 in 10) to be exceeded during a 50-year period. Linear interpolation between contours is intended.

Figure 5: Effective Peak Accelerations (as a fraction of gravity) for the United States of America (after Reference 3)

the maximum acceleration on firm ground that affects the behavior of sizeable bridges. Thus the effective peak acceleration ignores accelerations resulting from high frequency ground motion that have no impact on bridge response. The contours on the map indicate acceleration levels that are expected to be exceeded during a fifty year period with a probability of 10 percent.

2.3 NATURE OF EARTHQUAKE MOTIONS

The various types of waves that travel both within the earth and around its surface were described in Section 2.1. However, the interaction among these waves and the geologic strata were not discussed. It is this interaction, combined with the reflection and refraction of these waves by subsurface strata, that gives rise to the ground motion at a particular bridge site.

The actual speed of travel for all waves depends on the density and elastic properties of the material through which they pass. For granite, P waves travel at about 3.5 miles/second, and S waves at about 2 miles/second. In most earthquakes, the P waves are felt first. Some seconds later, the S waves arrive with their vertical and horizontal motions. It is this wave motion that is particularly damaging to bridge structures.

When body waves travel through layers of rock in the earth's crust, they are reflected and refracted at layer interfaces. When reflection or refraction occurs, some of the energy of one type of body wave is converted into waves of the other type. Thus the motion at a distance from the epicenter is a complicated mixture of the various types of waves.

There is considerable evidence that earthquake waves are affected by both soil conditions and topography. For example, in weathered surface rocks, in alluvium and saturated soils, the amplitude of seismic waves may be either decreased or increased as they pass to the surface from more rigid basement rocks. Also, at the top or bottom of a ridge, shaking may intensify, depending on wave incident direction and wavelength.

2.3.1 Prevalent Directions

The total motion at a site during an earthquake is highly irregular. Two horizontal components and one vertical component are present in varying amounts depending on the site. Studies have indicated that "principal" axes exist for the horizontal motion, accelerations being largest when measured in the direction of the major principal axis. This axis is directed approximately towards the epicenter and is approximately constant in direction during the duration of strong shaking. Generally, vertical motions are less than horizontal motions and the time of occurrence of the maximum vertical movement does not necessarily coincide with that of the maximum horizontal movement.

2.3.2 Frequencies and Spectra

A recorded earthquake acceleration history may be decomposed into a series of sinusoidal waves each with a different amplitude and frequency. A plot of these amplitudes against the corresponding frequencies is known as a Fourier spectrum. High Fourier amplitudes in a particular frequency range indicate large amounts of energy in the earthquake within that frequency range. Typical earthquakes are rich in frequencies from less than 0.5 Hz to about 20 Hz. Vertical motions have higher frequency

components than do horizontal motions.

An alternate method of characterizing the energy content of an earthquake is to use the earthquake's effect on a range of simple structures (called oscillators) with different natural frequencies of vibration. If the maximum response for each oscillator is plotted against its frequency, the resulting curve is a **response spectrum**. This curve is also a measure of the distribution of energy with frequency for a given earthquake. Response spectra are used extensively in seismic design and are further discussed in Section 3.4. The response spectrum for acceleration for the El Centro (1940) earthquake is shown in figures 14 and 15.

The amplitude of ground acceleration decreases with distance from the causative fault. The higher frequency components die out more rapidly than the lower frequency ones, so that frequency content is a function of distance from the fault.

2.3.3 Long Period Waves

At large distances (greater than 300 miles) from major earthquakes, very long period (10 to 20 seconds) surface waves may be experienced. The possibility for such waves should be evaluated on a case by case basis for sites where long structures on many supports are proposed. Long period waves could pose problems for this type of structure. Different input motions at each pier may give rise to large relative deformations in the superstructure and various substructures. Bridges on tall piers may also be adversely affected by long period waves.

An example of the damage that can be caused by unexpected long period waves is the destruction of an area of Mexico City by the Guerrero-Michoacan earthquake (1985) which occurred 250 miles from the city, off the coast of Mexico. The damaged area of the city was built on a drained lake bed that comprised very large deposits of soft muds. The dominant period of these deposits was about 2 seconds and these were excited by the arrival of the long period 2 second waves from the distant earthquake. Resonance effects occurred which caused large ground movements (16 ins peak-to-peak) and which in turn caused substantial damage to buildings with fundamental periods of vibration near 2 seconds (e.g. those in the 8-20 story range).

2.4 EARTHQUAKE PROBABILITY STUDIES

In order to develop appropriate seismic design forces, information concerning the probabilities of occurrence of earthquakes with varying magnitudes in the region of the site must be examined. For most sites, this information is adequately contained in codes or related documents. However, for major structures or for sites with unusual geological features, site specific studies may be warranted.

For seismically active regions, information on the frequency of occurrence of earthquakes with various magnitudes is available from historical records. So too is information concerning likely maximum motions, and their frequency content. In areas with limited seismic data, the use of sound engineering judgement by experts in seismology and geotechnical engineering is required.

A general warning concerning the use of results of probability studies must be made. Such results are only as good as the data upon which they were based. Reliable results

can be obtained from probability studies if the data base contains **quality** data in sufficient **quantity**. If such results are interpreted by experts, in light of general historical seismicity, meaningful conclusions can be reached.

2.4.1 Magnitude-Frequency Relationships

One of the most valuable studies is to determine the largest earthquake likely to occur near the site during the life of the structure being designed.

To provide an answer, plots of frequency of occurrence versus magnitude can be constructed from historical data near the site. Several investigators have done precisely this, both for localized areas of high seismicity and for the world as a whole. Gutenberg and Richter [reference 12] proposed an empirical relationship between frequency of occurrence and magnitude which takes the form:

$$\log N = a - b M \tag{2}$$

where N is the number of shocks of magnitude M or greater per unit time, and a and b are seismic constants that vary depending on the region's seismicity. These constants are derived by methods of statistics applying curve fitting techniques to observed data.

Table 1 summarizes the frequency of occurrence of earthquakes of a particular magnitude on a worldwide basis. It is seen that more than a hundred thousand earthquakes of magnitude 3 or greater occur every year throughout the world.

Table 1: Worldwide Earthquakes Per Year (from Reference 7)

MAGNITUDE M	AVERAGE NUMBER ABOVE M
8	2
7	20
6	100
5	3,000
4	15,000
3	More than 100,000

2.4.2 Peak Ground Motions

When determining peak ground motions at a given site for a given return period, the attenuation or decay of motion with distance from the fault is of prime importance. It is common to express the peak ground acceleration and velocity in terms of magnitude and epicentral distance or focal distance. The focal depth should be estimated from other earthquakes on the same fault or in the same region, and the epicentral distance should be measured perpendicular to the fault. There exist a great number of decay expressions, based on different amounts of data of different quality, from measurements all over the world. As might be expected, there is a very large scatter in the predictions

of such relationships, and only the more recent ones, which give statistical evaluations of the scatter of the data, should be used with confidence.

In fact, large scatters in peak acceleration as a function of epicentral distance are apparent from different sites in the same earthquake.

Perhaps the most broadly based relationship between magnitude, distance and peak acceleration is that from the work of Donovan [reference 13]:

$$a = \frac{1080\ e^{0.5M}}{(R + 25)^{1.32}} \tag{3}$$

where a is the mean peak acceleration in cm/sec/sec, R is the focal distance in km, and e is the exponential constant (2.7183). The equation is for the mean of 678 acceleration values of the Western USA, Japan and Papua New Guinea, and represents a conservative estimate of acceleration for sites with 20 feet or more of soil overlying rock.

Another attenuation relationship based on recent data is that of Esteva [reference 14], who gives the following expressions for peak ground acceleration, a in cm/sec/sec and velocity, v in cm/sec as:

$$a = \frac{5600\ e^{0.8M}}{(R + 40)^{2}} \qquad v = \frac{32\ e^{M}}{(R + 25)^{1.7}} \tag{4}$$

where e, M and R are as defined for equation (3).

These two expressions are based on California data and are valid for focal distances greater than 10 miles. It should be noted that attenuation relationships are generally inappropriate for small epicentral distances (less than 10 miles). Sites in this region require special consideration, because the state-of-the-art is still rather limited.

If equations (3) and (4) are plotted, families of curves are obtained for various earthquake magnitudes which relate peak acceleration to focal distance. An example of such a set of curves is shown in figure 6.

The prediction of peak ground displacement is not as reliable as that for acceleration or velocity. This is because displacements are usually calculated from a double integration of the acceleration trace and are subject to numerical errors. However, a rough estimate of ground displacement may be made from the empirical relationship of Newmark and Rosenblueth [reference 15]:

$$5 < \frac{ad}{v^2} < 15 \tag{5}$$

where d is the required displacement (units must be consistent with those used for acceleration, a and velocity, v). The lower bound for this non-dimensional ratio (5)

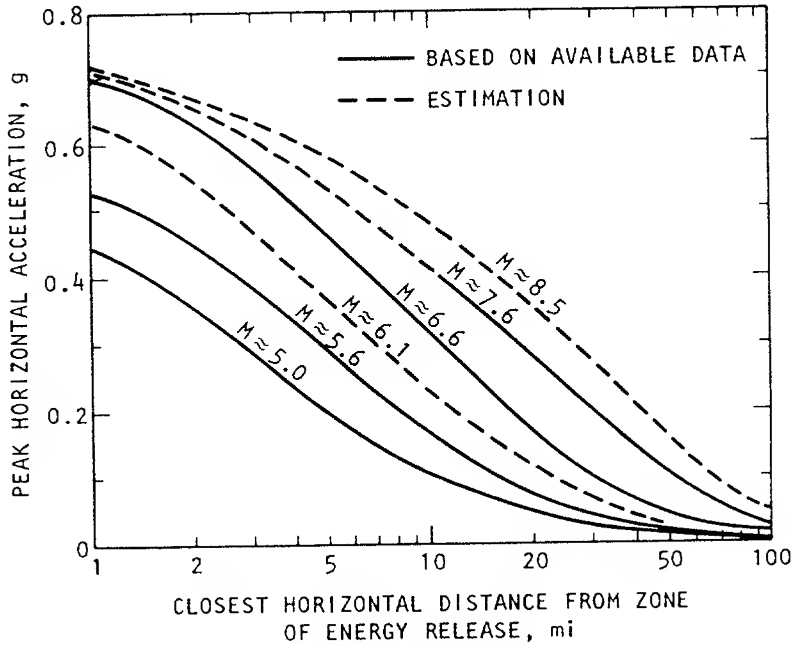


Figure 6: Average Values of Maximum Accelerations in Rock (from Reference 16)

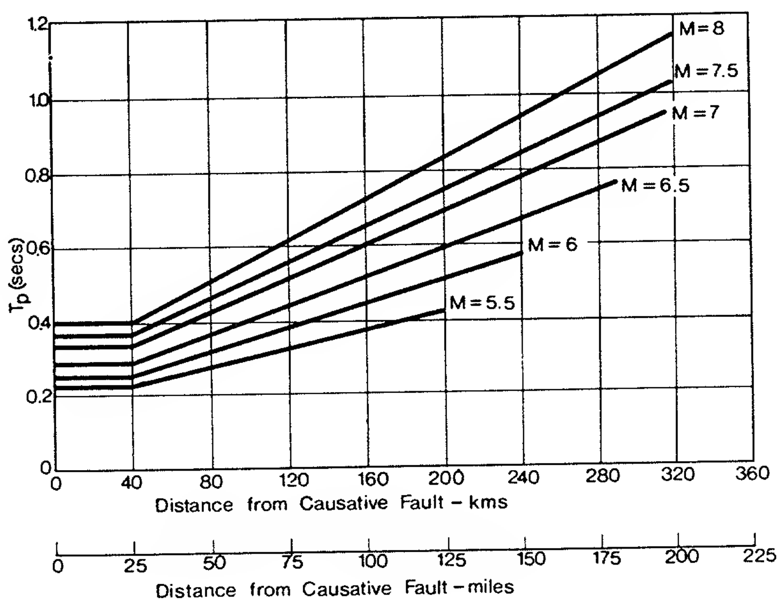


Figure 7: Predominant Periods (T_p) for Maximum Accelerations in Rock (from Reference 16)

is appropriate for large epicentral distances (greater than 60 miles) and the upper bound (15) is appropriate for small distances.

2.4.3 Frequency Content

In addition to peak ground motion parameters, the frequency content of the motion is vital in the study of bridge response to earthquakes. It is thought that frequency content, especially close to the epicenter, is a function of the type of rupture initiating motion. It is also a function of magnitude.

There is a distinct tendency for the predominant period of the motion to lengthen with distance from the epicenter. This is caused by shorter period motions decaying more rapidly with distance than long period motions. This effect is shown graphically in figure 7.

While the predominant period is a function of distance from the fault, the frequency content in general is also a function of the geologic structure of the site. Generally, sites with softer materials will have motions containing more long period components than will sites with stiffer materials.

2.5 DESIGN EARTHQUAKE

The design earthquake is usually defined in terms of a **design spectrum**. This can be estimated from the maximum ground motion parameters, together with some knowledge of the local soil conditions. However, it is important to realize that the amount of detailed knowledge of future earthquakes implicit in a design spectrum is limited, and not to be misled about the current state of understanding of earthquakes.

As discussed in detail in section 3.4, design spectra consist of a set of curves, each a function of structural period or frequency, with one curve for each indicated value of structural damping. It is important to recognize the distinction between a response spectrum and a design spectrum. A response spectrum is a set of jagged curves, one for each specified value of damping, giving some maximum response as a function of period **for a given ground motion**. The design spectrum on the other hand, is used to specify the level of seismic design force or displacement as a function of period and damping. It represents response to a range of possible earthquakes at a particular bridge site. It does not represent structural response to a single earthquake.

The shapes of design spectra are sometimes determined by smoothing a response spectrum of a recorded event, or by averaging the response spectra of several similar recorded events. In other cases, the determination of the shape of the design spectrum is more complex, for the design spectrum may have to reflect the shaking potential from different types of earthquakes. For example, in an area with one major fault and several lesser ones, the short period portion of the spectra may be governed by close earthquakes with lower magnitudes, while the long period end would be controlled by a major event on the (more distant) major fault.

2.5.1 Local Geology and Soil Conditions

In order to specify consistent levels of structural capacity for structures having different natural periods, the design spectra should reflect the relative intensities of ground motion expected at different frequencies. Several sets of standard spectral shapes have emerged over the years, as a result of many different studies. Typically, only the overall amplitude is changed from site to site, the spectral shape remaining constant.

The following local geological features can affect site response.

- dimensions of soil deposit overlying bedrock
- slope of bedding planes and of sedimentary deposits
- changes of soil types horizontally across a site
- topography of both soil and bedrock
- ridges
- potential for liquefaction
- soil types and conditions of deposit

The effect of soil type on acceleration spectra is discussed in further detail in chapter 6. Figure 57 of section 6.2 shows that higher accelerations are generated in medium period structures on softer soils. For example, a bridge with a fundamental period of 1.0 sec will experience a four-fold increase in acceleration if moved from a rock site to one on soft clay.

One problem that frequently arises is how to adjust the design spectra to take account of possible influences of local geology and soil conditions. The most important point to remember is that it is not generally justified to spend a large effort in tailoring the shape of the design spectra to fit a given site. Where this effort is warranted, standard shapes are generally used as the starting point and minor deviations from these yield site-specific spectra.

Usually, detailed field data concerning the effects of local conditions are not generally available, and in these circumstances, ground motions at some depth (bedrock or firm soil) are estimated and then propagated up to the surface using computer modelling techniques. However, results from such analyses should be used with caution for they can be very sensitive to the assumed soil properties and the motion at depth. Generally, they should be used as secondary guides to support observations made in recorded accelerograms from similar geologic and seismologic conditions. The observed behavior should be used as a primary guide in reshaping spectra for a site-specific application.

CHAPTER 3 BASIC DYNAMICS

Dynamic loads are loads that vary with time. Structural dynamics is the study of structural response to these loads. Bridges are subject to several kinds of dynamic loads ranging from wind and vehicle effects to earthquakes. Response to these loads can be markedly different to that under static loads and indeed bridges that have repeatedly withstood static loads have collapsed under dynamic loading of similar or less magnitude.

The essential difference between static and dynamic loads is the time varying nature of the dynamic loads. If the frequency content of the applied load is close to the frequency of vibration of the bridge, the structure will amplify the loading and large and potentially destructive forces will be generated within the bridge. Essentially, problems arise when frequency matching occurs. This is the basis of all resonance phenomena. Load which is applied very slowly causes response which is virtually identical to static loading. On the other hand, cyclic load which is applied very rapidly (i.e. with high frequency) has negligible effect on a structure. Amplification of load (sometimes also known as response magnification) only occurs when the rate of application (frequency) of load is near one of the natural frequencies of one of the modes of vibration for the bridge. Different bridges will therefore respond differently to the same load because their natural frequencies will be different. Since typical highway bridges vibrate with frequencies in the range 0.5 to 20 Hz and typical earthquakes have the same frequency content (section 2.3.2), there is a very real possibility that frequency matching will occur between a bridge and the ground during an earthquake.

It is clearly important to be able to analyze a bridge for dynamic loads and the intent of this chapter is to outline basic principles of bridge dynamics. Rigorous treatment of the subject rapidly becomes mathematically complex and today there are several computer programs which take care of these complexities. However, a physical appreciation of bridge dynamic behavior cannot be obtained from the use of computer software or from the pages of an applied mathematics text on differential equations. Instead, the study of simple models, which approximate the behavior of actual bridges, is recommended. Equilibrium equations become equations of motion from which natural frequencies may be calculated and member forces and deflections may be estimated. The primary factors influencing response (weight, stiffness and damping) are more easily identified from these models but their limitation must also be clearly understood. Consequently, some background in engineering mathematics is necessary and, in fact, it is assumed in the treatment given below.

If more detail is required in these areas, several good textbooks are available, for example, references 17, 18 and 19.

3.1 BRIDGE MODELLING

Bridges are assumed to respond to earthquake loads in one of two horizontal directions: transverse and longitudinal (span-wise) respectively. Response in the vertical direction is usually ignored for simple bridges provided restraint against relative vertical movement is considered at the detail design stage. Actual seismic response will probably be in a direction somewhere between these two principal horizontal directions and to account for this possibility, combination of response from each direction is recommended. For the present, it is sufficient to consider behavior in one or the other direction assuming no interaction.

The simplest of all bridge models is the single mass model restrained by an elastic spring. In this model, the mass is assumed to have one degree of freedom; i.e., it can move in only one direction. This assumption is reasonably accurate for the longitudinal response of straight continuous bridges. However, its reliability for transverse behavior is conditional on several factors and it may not be satisfactory for complex bridges. Nevertheless, for typical highway structures, the single degree of freedom (sdof) model is sufficiently accurate for design purposes, especially if care is taken with the selection of the equivalent mass and stiffness parameters. The limitations on this sdof model and the analysis of complex bridges is discussed in section 3.5.

3.1.1 Example Bridge

To illustrate the spring-mass model a bridge example is shown in figure 8a. The two-span bridge has seat-type abutments with sliding bearings to permit longitudinal movement but the superstructure is monolithic with a multi-column bent. Transverse shear keys are provided at each abutment to prevent transverse movements across the abutments and to lock the superstructure to the abutments in this direction. The abutments and the foundation structure below the pier are all assumed to be rigid in both directions.

3.1.2 Longitudinal Model

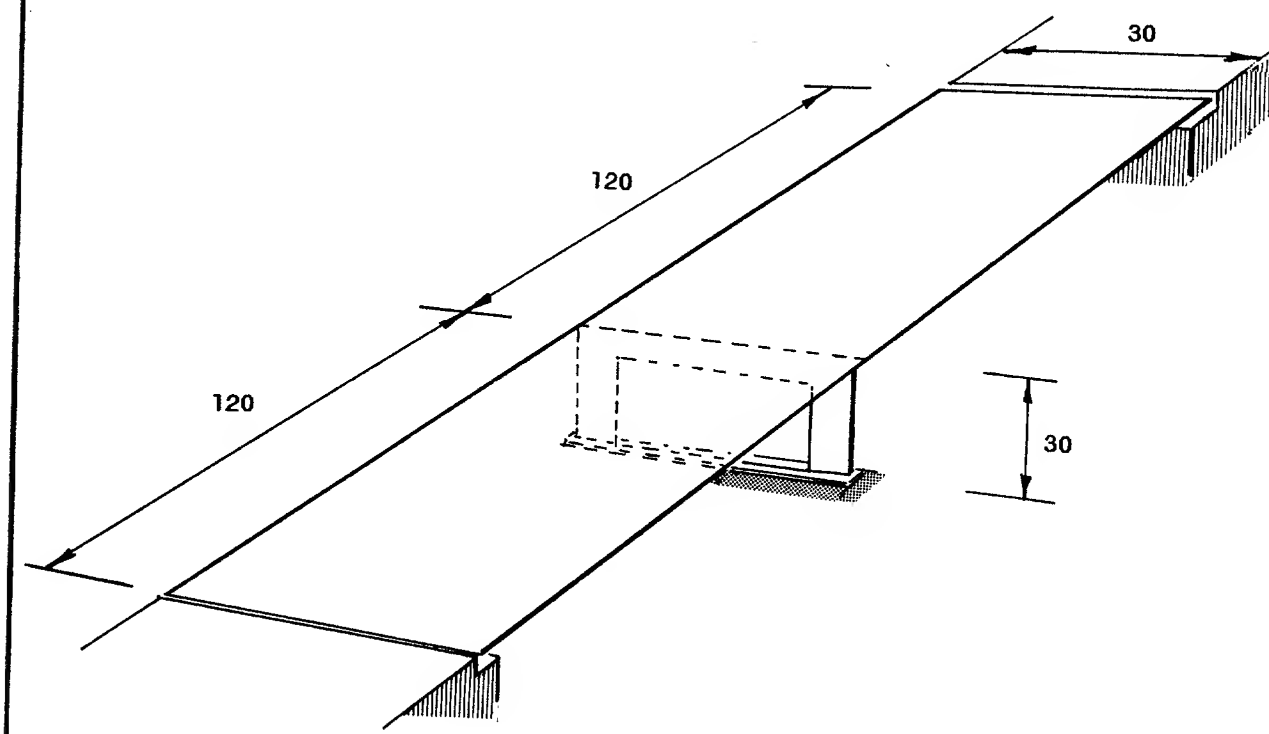
Figure 8b shows an elevation of this bridge and its equivalent spring-mass system.

The mass, M_L, represents the total mass of the superstructure and perhaps a portion (one-third) of the mass of each column.

Usually the superstructure is the heaviest component in a bridge, by a wide margin, and for the purpose of these simplified models it is common to neglect the mass contributions from other components.

The longitudinal movement of the superstructure mass, d, becomes the displacement degree-of-freedom of the spring-mass system.

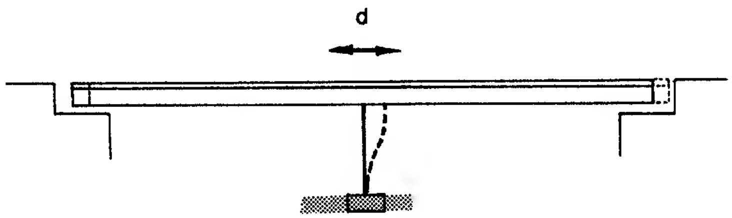
The spring constant (stiffness, K_L) is given by the sum of the stiffnesses of all the structural components effective in this direction. Since free sliding is presumed to occur at the abutments, only the columns contribute to K_L. Expressions for column stiffness for different end conditions are summarized in figure 47.

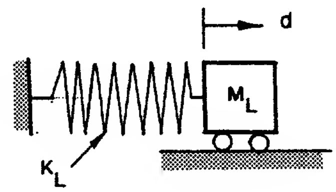


span lengths (both)		120 ft
dual column bent height		30 ft
I_c,	moment of inertia (each column, both directions)	25 ft^4
	dead load (superstructure)	$7^K/ft$
I_s,	moment of inertia (superstructure)	4000 ft^4
E,	elastic modulus	432,000 Ksf

(a) Geometry and Properties

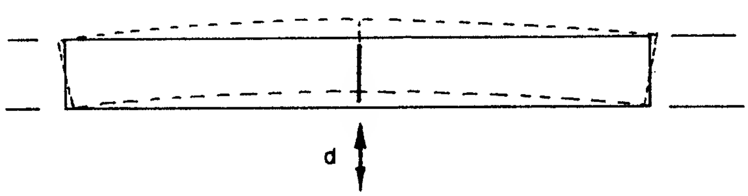
Figure 8: Basic Bridge Dynamics, Bridge Example

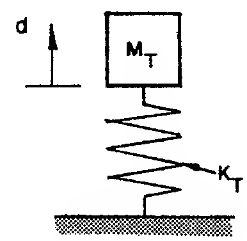




K_L stiffness in longitudinal direction
= stiffness of bent (only) $= 2(12)EI_C/h^3 = 9600$ K/ft
M_L = total mass of superstructure $= 7(240)/g = 1680/32.2$ $Ksec^2/ft$

(b) Equivalent Model for Longitudinal Response





K_T = equivalent stiffness in transverse direction
M_T = equivalent mass in transverse direction

(c) Equivalent Model for Transverse Response

Figure 8: Basic Bridge Dynamics, Bridge Example (continued)

3.1.3 Transverse Model

Figure 8c shows a plan view of the bridge and its equivalent spring-mass system.

The effective mass, M_T, appropriate to this model needs to be calculated according to the method of analysis used. It can range from the total superstructure mass to the tributary mass appropriate to each pier as discussed below for each of the methods of analysis.

The transverse movement of this effective mass, d, becomes the displacement degree-of-freedom of the spring-mass system.

Since the abutments are both assumed to be perfectly rigid (infinite lateral stiffness) the superstructure does not displace as a rigid body (as in the longitudinal case) but rather deflects as a beam spanning from one fixed abutment to the other.

The lateral stiffness, K_T, is then a combination of the in-plane flexural stiffness of the superstructure and the lateral stiffness of the columns. Various means are used to determine this effective stiffness and some of these are described below.

The first and simplest is to neglect the continuity in the superstructure altogether and assume each bent and each abutment acts independently of each other. The total mass of the superstructure is then divided up into discrete concentrated masses lumped above each pier or abutment according to the tributary length of the superstructure. This idealization gives the model the appearance of a number of lollipops standing in line (large concentrated masses supported on flexible piers) and is the basis of the so-called **lollipop method**. In this case M_T is given by the tributary mass and K_T is just the lateral stiffness of the bent acting alone. This model will clearly underestimate the lateral stiffness of the bridge because it neglects the inplane bending stiffness of the deck. It therefore has limited application. The method was recommended in the earlier editions of the AASHTO Standard Specifications but it has now been replaced by the more realistic uniform load method.

The **uniform load method** calculates a value for K_T by applying a uniform lateral load (w) to the bridge and finding the maximum deflection (Δ).

Then: $$K_T = wL/\Delta \qquad (6)$$

where L is total length of bridge.

The analysis for this load case includes all structural members contributing to the lateral stiffness and foundation effects when these are known. Such a calculation may require a computer program for solution, especially if the structure is at all complex. But for the above bridge example it is possible to perform the analysis by hand using the superposition of load cases to obtain the desired result. The equivalent mass, M_T, to be used in the uniform load method is assumed equal to the total superstructure mass as in the longitudinal direction. This method is recommended in both the AASHTO and CalTrans Standard Specifications [references 1 and 2].

In one refinement to the uniform load method, the transverse mode shape is used to obtain the equivalent lateral stiffness. In the so-called **generalized coordinate method**, this mode shape is assumed to be given by a half sine wave and the mass

distribution is again approximated by lumped values at each bent position. The validity of the assumed mode shape is dependent on the ratio of stiffnesses between the substructures and superstructure (defined as a stiffness index). If the superstructure dominates the stiffness, the method can give better results than the uniform load method.

Probably the most accurate of the equivalent static force methods is the **single mode spectral analysis** method recommended in the AASHTO Guide Specifications for seismic design [reference 4]. It uses the uniform load technique to generate an approximate mode shape and is therefore not as dependent on the stiffness index for accuracy. The actual mass distribution is also used in this method which generates a more realistic value for the equivalent mass, M_T. It ensures more accurate values for member forces which are calculated from the seismic coefficient and period of vibration. This method is described in detail in chapters 7, 8 and 10.

3.2 EQUATIONS OF MOTION

3.2.1 Undamped, Free Vibration

Given an equivalent model for the bridge structure, it now remains to write the equation of motion and solve for displacement and frequency. As figure 9a illustrates, equilibrium is used to obtain the desired equation which takes the form of a second-order differential equation in displacement. This may not be obvious from the boxed equations in figure 9a because of the notation used for acceleration and displacement. However, if x is substituted for d, then velocity (which is the rate of change of displacement with time) is given by dx/dt and acceleration (which is the rate of change of velocity with time) is given by d^2x/dt^2. If these first and second order derivatives are expressed as $\dot{x}$ and $\ddot{x}$ respectively, the equilibrium equation in figure 9a becomes

$$M\ddot{x} + kx = 0 \tag{7}$$

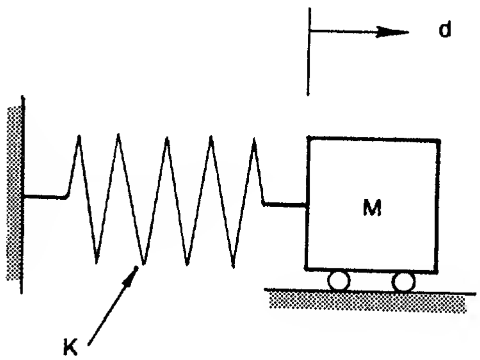
which is clearly an equation in x, the displacement variable.

To complete the solution, initial boundary conditions must be specified, giving a harmonic expression for displacement. The period of vibration is then shown to be:

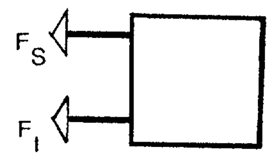
$$T = 2\pi \sqrt{M/K} \tag{8}$$

3.2.2 Damped, Free and Forced Vibration

So far in this discussion, damping has not been considered. However all structural systems exhibit damping to varying degrees and its effect on dynamic response is generally beneficial. Structural damping is assumed to be viscous by nature, which means that damping forces are directly proportional to velocity. If the damping coefficient, which relates force to velocity, is sufficiently large, it is possible to totally suppress the oscillation of a spring-mass system. The minimum value of this coefficient that just prevents oscillatory motion, is called the critical value. A convenient measure of damping is then possible by comparing actual values of the damping coefficient with this critical value. This ratio is frequently expressed as a percentage and typical values for bridge structures fall in the range 2 to 10 percent. Although these might appear to be small values, they are nevertheless very important in controlling peak displacements and forces, especially near resonance and for bringing a bridge to rest



SPRING-MASS SYSTEM



FREE-BODY DIAGRAM FOR MASS

EQUILIBRIUM REQUIRES $\boxed{F_I + F_S = 0}$

where F_I = Inertia force (reluctance of mass to accelerate and acts to oppose motion)
= mass x acceleration
= M.a

and F_S = spring force
= stiffness x displacement
= K.d

Substitution gives $\boxed{Ma + Kd = 0}$

Initial conditions must now be specified

For example, if at time $t = 0$, $d = D$ (initial displacement)

then solution to equation of motion will be:

$d = D\cos pt$

where p = natural frequency of vibration (in radians/sec.)

$= \sqrt{K/M}$

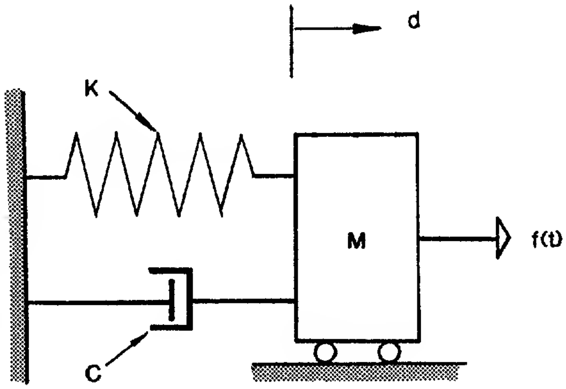
THEREFORE

1. period of motion is given by $\boxed{T = 2\pi \sqrt{M/K}}$

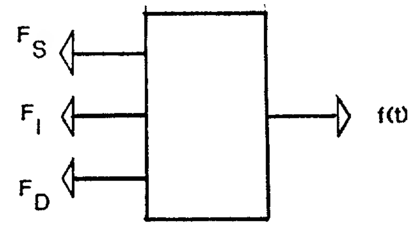
2. velocity $v = -pD\sin pt$ and acceleration $a = -p^2D\cos pt$

3. maximum velocity = p x maximum displacement
maximum acceleration = p^2 x maximum displacement

Figure 9a: Equations of Motion for Free Vibration of a Single Mass



SPRING-MASS DASHPOT SYSTEM



FREE-BODY DIAGRAM FOR MASS

EQUILIBRIUM REQUIRES

$$\boxed{F_I + F_D + F_S = f(t)}$$

where F_I = inertia force = Ma
F_D = damping force = damping coefficient * velocity = cv
F_S = spring force = Kd

Substitution gives

$$\boxed{Ma + cv + Kd = f(t)}$$

Initial conditions and the forcing function must now be specified

FOR EXAMPLE:

f(t) may be a harmonic load applied directly to the mass

then $f(t) = F\sin\omega t$

where F is the amplitude (maximum value) of the force
and ω is the frequency of the forcing function (radians/sec.)

or f(t) may be a harmonic load induced by harmonic base excitation

then $f(t) = -Ma_g = -MA_g\sin\omega t$

where a_g is the base (ground) acceleration
A_g is the amplitude of base acceleration
ω is the frequency of ground movement (rads/sec.)

or f(t) may be any other periodic or random function of time

Figure 9b: Equations of Motion for Forced Vibration of a Single Mass

	UNDAMPED
FREE VIBRATION	$v(t) = \rho \cos(\omega t - \theta)$ $T = 2\pi\sqrt{M/K}$ where ρ and θ are given by initial release conditions
FORCED VIBRATION (a) harmonic load applied directly to mass i.e., $p(t) = p_o \sin\bar{\omega}t$	assume "at rest" initial conditions then solution is $v(t) = \frac{p_0}{k}\frac{1}{1-\beta^2}(\sin\bar{\omega}t - \beta\sin\omega t)$
(b) harmonic base excitation i.e., $p(t) = -M\ddot{v}_g$ $= -M\ddot{v}_{go}\sin\bar{\omega}t$	assume "at rest" initial conditions then solution is $v(t) = \frac{v_{g0}\beta^2}{1-\beta^2}(\sin\bar{\omega}t - \beta\sin\omega t)$
(c) nonperiodic load $p(t) = p(\tau)$ at time $t = \tau$	assume "at rest" initial conditions then solution is (Duhamel's integral): $v(t) = \frac{1}{m\omega}\int_0^t p(\tau)\sin\omega(t-\tau)\,d\tau$

NOTATION

The notation used in the above Table is that of Clough and Penzien: Dynamics of Structures (McGraw-Hill, 1975). The equivalent notation used in this Manual is as follows:

Above Table	Definition	This Manual
β	frequency ratio	r
θ	phase angle	not used
ξ	damping ratio	n
ρ	maximum displacement	D
ω	natural frequency of bridge	p
ω_D	damped natural frequency of bridge	not used
$\bar{\omega}$	frequency of forcing function	ω

(a) Undamped Systems

Figure 10: Solutions to Equations of Motion for Single Masses

	DAMPED
FREE VIBRATION	$$v(t) = \rho e^{-\xi\omega t} \cos(\omega_D t - \theta)$$ where ρ and θ are given by initial release conditions
FORCED VIBRATION (a) harmonic load applied directly to mass i.e., $p(t) = p_o \sin\bar{\omega}t$	assume initial transient terms have been damped out then steady state solution is $v(t) = \rho \sin(\bar{\omega}t - \theta)$ $\rho = \frac{p_o}{k}[(1-\beta^2)^2 + (2\xi\beta)^2]^{-1/2}$ $\theta = \tan^{-1}\frac{2\xi\beta}{1-\beta^2}$
(b) harmonic base excitation i.e., $p(t) = -M\ddot{v}_g = -M\ddot{v}_{go}\sin\bar{\omega}t$	assume initial transient terms have been damped out then steady state solution is $v(t) = \rho \sin(\bar{\omega}t - \theta)$ $\rho = \frac{m\bar{\omega}^2 v_{go}}{k} D = v_{go}\beta^2 D$ $D \equiv \frac{\rho}{p_o/k} = [(1-\beta^2)^2 + (2\beta\xi)^2]^{-1/2}$
(c) nonperiodic load $p(t) = p(\tau)$ at time $t = \tau$	assume "at rest" initial conditions then solution is (Duhamel's integral): $v(t) = \frac{1}{m\omega_D}\int_0^t p(\tau)e^{-\xi\omega(t-\tau)} \sin\omega_D(t-\tau)\,d\tau$

NOTATION

Above Table	Definition	This Manual
D	Response Magnification Factor	not used
k	spring stiffness	K_T or K_L
m	mass	M
p(t)	force as a function of time	f(t)
p_o	maximum value of p(t)	F_o
v(t)	displacement as a function of time	d(t)
$\ddot{v}_g$	ground acceleration	a_g
$\ddot{v}_{go}$	maximum value of v_g	A_g

(b) Damped Systems

Figure 10: Solutions to Equations of Motion for Single Masses (continued)

at the end of an earthquake. Since the frequency of vibration of a damped system is only slightly different from an undamped system, it is usual to neglect damping in frequency and period calculations unless it exceeds about 20 percent.

Equilibrium is again used to analyze the response of a damped structure by the introduction of the damping force into the equilibrium equation. This is illustrated in figure 9b.

If, instead of vibrating freely, the spring-mass system is forced to vibrate by a time varying external force, this force may also be included in the equilibrium equation, as shown in figure 9b. The forcing function may be applied directly to the mass itself or be generated by base excitation (the earthquake situation) but in either case the formulation of the equation of motion is the same.

Solutions to the equation now depend on the nature of the external force. If harmonic, then rigorous closed-form solutions are available [reference 17] and these are summarized in figure 10.

Before leaving this section on equations of motion it is worth noting that solutions for nonharmonic loads exist and are described in numerous textbooks such as reference 17. The most common formulation is called Duhamel's integral and this solution is also given in figure 10. Evaluation of the integral is rarely possible in a closed form, but numerous computer algorithms have been developed to perform this calculation.

3.3 PERIOD OF VIBRATION

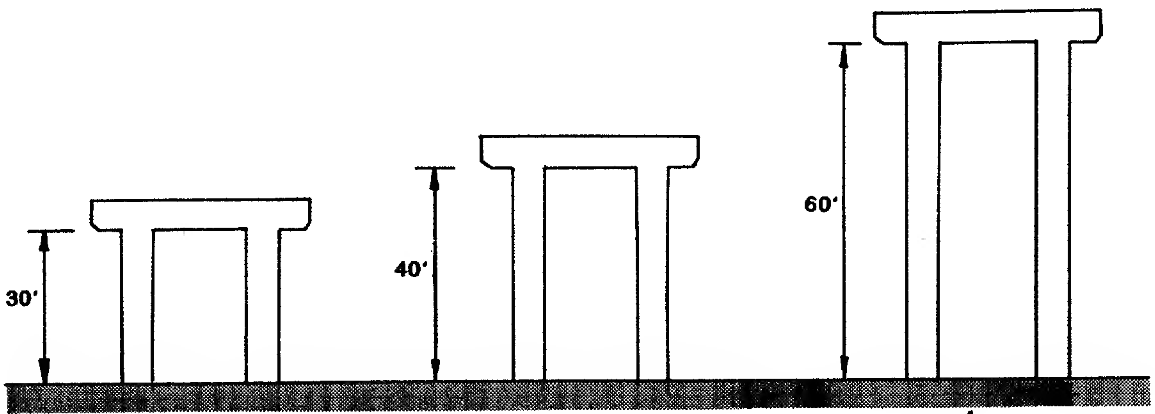
The term natural frequency is used to mean the frequency at which a bridge will vibrate freely, without being forced in any way. Free vibrations, as they are often called, are most commonly measured or calculated by initially deflecting the structure, releasing it to vibrate without interference, and recording the time it takes for the bridge to complete a given number of cycles. The number of cycles per unit time is then a measure of the natural frequency. The reciprocal of frequency is the time the bridge takes to complete one cycle of vibration. Called the period of vibration, this interval of time is used more commonly than frequency to describe a vibratory motion or a bridge's response to excitation. Equation (8) may be used to calculate this period for any vibrating system, including a bridge structure.

Figure 11 summarizes the periods of vibration for the example bridge. These have been calculated using equation (8) and substituting appropriate values for mass and stiffness. It is seen that the period in the longitudinal direction is longer than the transverse period because of the lack of abutment restraint in this direction. It will also be seen that, for this example, there is negligible difference in the various estimates for transverse period. Each method gives a similar value for the period but this is not generally true for all bridges since, as noted earlier, the relative contributions of stiffness (from the superstructure and substructure) affect the accuracy of the various methods.

The influence of pier height and superstructure weight on the period is also illustrated in figure 11. It is seen that as the pier height increases from 30 ft to 60 ft the period increases by almost a factor of 3. This is because the taller piers are considerably more flexible and these bridges have longer periods due to their inherent flexibility.

DIRECTION AND METHOD	M	K	PERIOD
longitudinal	1680/g	$9600^{K}/ft$	$2\pi\sqrt{1680/(32.2)(9600)}$ = 0.46 secs
transverse			
1. lollipop method	840/g	9600	$2\pi\sqrt{840/(32.2)(9600)}$ = 0.33 secs
2. uniform load method	1680/g	24,960	$2\pi\sqrt{1680/(32.2)(24960)}$ = 0.29 secs
3. generalized coordinate method	840/g	15,600	$2\pi\sqrt{840/(32.2)(156000)}$ = 0.26 secs

(a) Results for Example Bridge in Figure 8



MASS, M_T ($Ksec^2/ft$)	STIFFNESS, K_T (K/ft)		
	9600 (h = 30)	4050 (h = 40)	1200 (h = 60)
13.045	.232	.357	.655
26.090	.328	.504	.926
52.180	.463	.713	1.310
Units: Period in seconds; Pier height (h) in feet			

(b) Results for Different Column Heights and Superstructure Mass

Figure 11: Periods of Vibration for Different Bridges

Flexibility may be increased, and the stiffness reduced, by a variety of means which include (in addition to increasing the height), reducing the pier cross section and changing the structural type (from a bent to a single column for example). Conversely, bridges with stiff piers will have shorter periods, as will be the case for short-to-medium pier heights, heavy column cross sections and for wall and multicolumn bent piers.

A heavier superstructure also responds with a longer period as shown in figure 11. It is seen that a fourfold increase in weight will increase the period by a factor of two.

One inescapable conclusion from figure 11 is that light, stiff bridges have short (low) periods of vibration and respond to excitation swiftly with high frequency vibration. On the other hand, heavy, flexible bridges have long periods of vibration and respond to excitation sluggishly with low frequency vibration.

3.4 RESPONSE SPECTRA

Earthquakes subject structures to time-varying forces which, in turn, produce time-varying displacements and stresses within these structures.

From a design viewpoint, only the maximum value of displacement and stress is of interest--the variation with time of these quantities is of little consequence. For survival the structure must withstand the peak value whenever that may occur. Therefore an engineer need only know this one magnitude to successfully design a bridge and this information is made available in the form of response spectra.

It was observed in section 2.3.2 that it is possible to generate curves which give peak displacements for any structure subject to a given earthquake. These curves are called response spectra because they give the response (e.g., maximum displacement) of a wide spectrum of structures as defined by their frequency (or period) and damping ratio.

A useful introduction to this concept and to the immense value of these spectra can be obtained by first considering response spectra for bridges subject to simple harmonic motion.

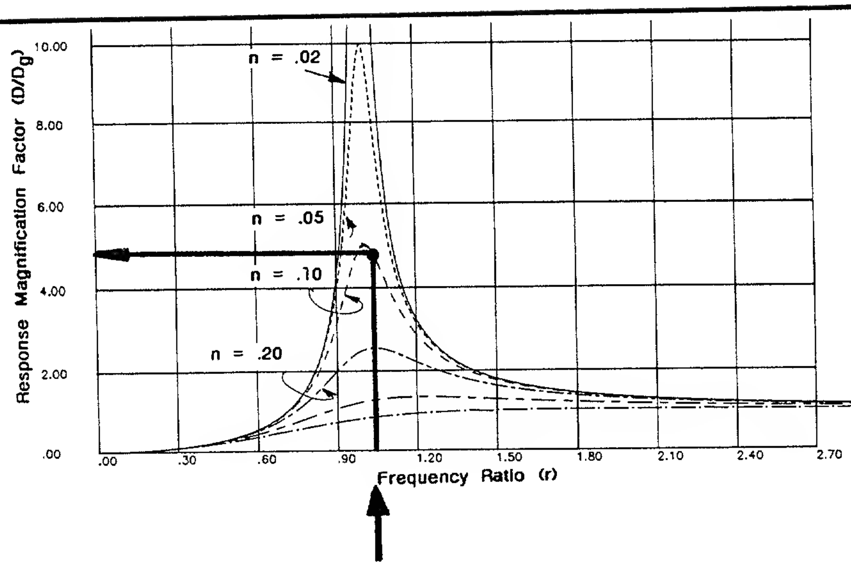
3.4.1 Response Spectra for Harmonic Excitation

Although real earthquakes are not harmonic or even periodic, a very important concept can be developed from the results for harmonic base excitation (figure 10). For example, if the expression: $\rho = v_{go}\beta^2 D$ (figure 10) is rewritten in the notation of this Manual and slightly rearranged, it becomes:

$$D \;/\; D_g = r^2 \;/\; \sqrt{(1-r^2)^2 + (2nr)^2} \tag{9}$$

where r is the frequency ratio ($=\omega/p$) and n is the damping ratio. Then if D/Dg is plotted against r, figure 12 is the result.

The vertical axis of this figure gives the peak displacement D of a single degree-of-freedom system (when subject to a sinusoidal earthquake of peak displacement D_g



Consider the example bridge (Figure 8) to be subjected to a harmonic base motion of amplitude 0.75 inches and frequency 3.5 Hz.

The above Response curves can be used to calculate the maximum displacement in the bridge.

STEP 1: Assume period of bridge is 0.3 seconds and calculate frequency ratio, r.

Natural frequency of bridge (p) $= 2\pi/T = 20.94$ rads/sec

Natural frequency of ground (ω) $= 2\pi\ (3.5) = 21.99$ rads/sec

therefore $r = \omega/p = 21.99/20.94 = 1.05$

STEP 2: Assume 10% equivalent viscous damping in the bridge, then for $r = 1.05$ and $n = 0.10$, the Response Magnification Factor $D/D_g = 4.72$

STEP 3: Maximum displacement, $D = 4.72\ D_g$
$= 4.72\ (0.75)$
$= 3.54$ inches

For comparison of performance, the Table below summarizes the responses of 6 bridges subject to the same sinusoidal motion. The bridges have periods of 0.3, 0.5 and 0.9 seconds and damping ratios of 2% and 10%. The sinusoidal motion is that used above (0.75 inches amplitude at 3.5 Hz).

Damping Ratio (n)		0.02			0.10		
Period (T)	**seconds**	.3	.5	.9	.3	.5	.9
Frequency (p)	rads/sec	20.94	12.57	6.98	20.94	12.57	6.98
Frequency ratio (r) = 21.99/p		1.05	1.75	3.15	1.05	1.75	3.15
Magnification factor (D/D_g)		9.95	1.48	1.11	4.72	1.46	1.11
Spectral displacement ($D = S_d$)	ins	7.46	1.11	.83	3.54	1.10	.83
Spectral velocity ($S_v = p.S_d$)	ins/sec	156.3	13.95	5.81	74.13	13.76	5.81
Spectral acceleration ($S_a = p^2.S_d$)	ins/sec^2	3272	175.4	40.56	1552	173.0	40.56
Spectral acceleration (S_a)		8.47g	.45g	.10g	4.02g	.45g	.10g

Figure 12: Displacement Response Spectra for Sinusoidal Base Motion

and frequency (ω) in terms of its natural frequency p for a range of damping ratios. Therefore this figure can be used in one of two ways:

either it will give the response (D) of a range of different single degree-of-freedom systems to the same sinusoidal earthquake (D_g, ω fixed)

or it will give the response (D) of one single degree-of-freedom system (p, n fixed) to a range of different sinusoidal earthquakes.

Note that to use these curves, the only information required to describe the structure is its frequency (or period) and its damping ratio (n). Actual values of mass, stiffness and damping coefficient are not required. Furthermore, these artificial sinusoidal earthquakes are completely described by peak ground displacement and frequency.

Figure 12 gives an example of the use of these response curves to predict the behavior of six bridges (including the example bridge of section 3.1.1) to sinusoidal ground motion. These bridges have periods of 0.3, 0.5 and 0.9 seconds and damping ratios of 2 percent and 10 percent.

It is seen that damping always reduces the response but that its effect is greatest when the frequency ratio is close to unity. For the 0.3 second bridge, this ratio is 1.05 and an increase in damping from 2 to 10 percent, reduces the maximum displacement by more than a factor of two--from almost 10 inches to about 4-3/4 inches.

Also demonstrated in this table is the reduction in response as the period of the bridge moves away from the period of the ground motion (0.29 seconds), i.e. as the frequency ratio increases above 1.0. Resonance effects are evident when the frequencies (periods) are matched, or nearly so. This is also graphically demonstrated in the response curves at the top of figure 12.

Since both of the above uses of the same set of curves involve the determination of **response** for a range or **spectrum** of different circumstances, it now becomes clear why these curves are called "response spectra." They would be extremely useful in design if sinusoidal earthquakes of constant amplitude and frequency, were encountered. However, the value of these spectra for predicting maximum dynamic response led to the development of similar curves for real earthquakes, as described below, and these have become the basis of modern seismic design.

Figure 12 is in fact a **displacement spectrum** for a sinusoidal earthquake. It is also possible to generate similar curves which give peak accelerations and peak velocities for sinusoidal earthquakes and these are called **acceleration** and **velocity response spectra**, respectively. However, these other spectra have not been plotted here because of the limited interest in sinusoidal earthquakes.

3.4.2 Response Spectra for Earthquake Excitation

To generate spectra for real earthquake time histories, Duhamel's integral is commonly used and maxima values recorded and later plotted against frequency (or period) and damping ratio. Because there is no closed-form solution to the integral equation each point on each curve in the spectra is computed numerically. A very large number of calculations are required for this process and the first spectra were calculated using

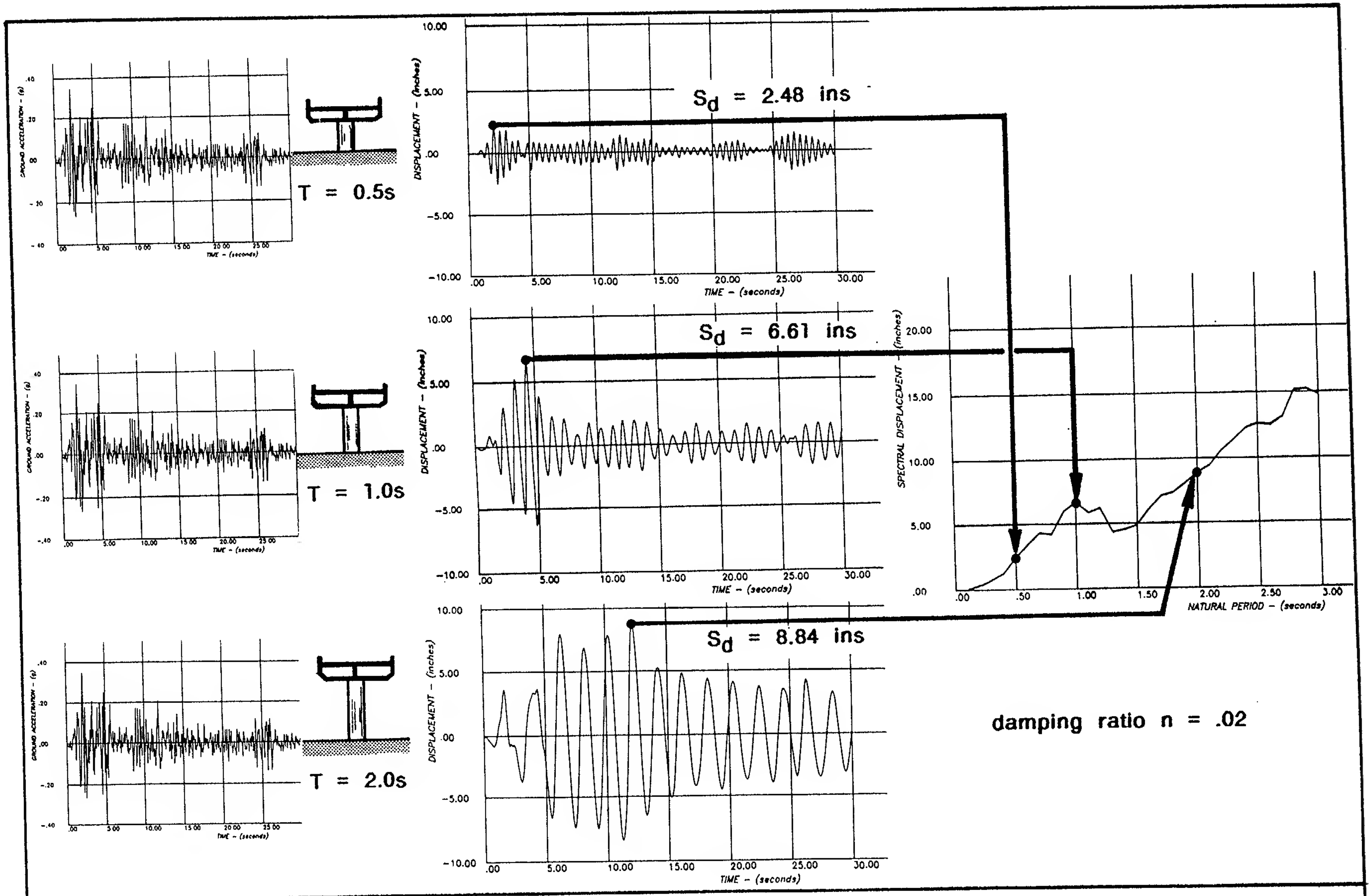


Figure 13: Development of the Displacement Response Spectrum for the El Centro (1940 NS) Earthquake (after Reference 19)

analog computers. Today the digital computer is routinely used to generate these curves.

The numerical process by which these spectra are calculated is illustrated in figure 13. A single degree-of-freedom structure with period, T, and damping ratio, n, is subjected (numerically) to a given earthquake time history. In figure 13 the NS component of the 1940 El Centro earthquake is used to excite a bridge with a period of 0.5 sec and 2 percent damping ratio. Duhamel's integral is evaluated at, say, one-hundredth-of-a-second intervals throughout the duration of the earthquake, giving a complete time history of displacement, velocity and acceleration of the mass. These time histories are scanned and the maximum values saved (2.48in in figure 13). The remainder of each record is discarded. These maximum values of displacement, acceleration and velocity are then plotted against period and damping ratio, giving one point on each of the three spectra. The whole process is then repeated for a new structure (period and damping ratio) and another point plotted on each spectra. In figure 13 the analysis of two further bridges is illustrated. These have periods of 1.0 sec and 2.0 sec respectively. The damping ratio is maintained at 2 percent. Maximum displacements of 6.61 ins and 8.84 ins are obtained and plotted against the corresponding periods. The procedure is continued until there are sufficient points to construct the required curves. To obtain accurate spectral curves several hundred structures may need to be analyzed in this way, which implies a substantial amount of calculation. Furthermore, new spectra need to be produced for each earthquake of interest.

No two displacement (or velocity or acceleration) response spectra are identical since each earthquake is itself slightly different in amplitude and frequency content.

However, there are certain trends which have been observed as follows:

> **acceleration spectra** tend to reach a maximum at about a period of 0.5 sec., then steadily reduce with increasing period
>
> **velocity spectra** also tend to reach a maximum at about a period of 0.5 sec., but then remain constant with increasing period
>
> **displacement spectra** show steady increases up to periods of 3 seconds, but then are expected to remain constant; reliable experimental confirmation in the long period range is not yet available.

Figure 14 shows the acceleration response spectra for the 1940 El Centro earthquake. From this spectra it is possible to determine the maximum acceleration that the example bridge in section 3.1.1 would have had to withstand to survive the El Centro earthquake.

Assuming 2 percent damping and a period of 0.5 secs. the peak acceleration is a little under 1g. This means that at some point in time during the earthquake the bridge would have been subjected to a horizontal acceleration of 1g. This is equivalent to its full weight acting horizontally. Since it is generally uneconomic to design a bridge to withstand a horizontal force equal to its self weight, damage to the substructure is to be expected. However, as soon as the structure starts to deteriorate, its stiffness drops, its natural period lengthens and possibly its damping increases due to inelastic deformation.

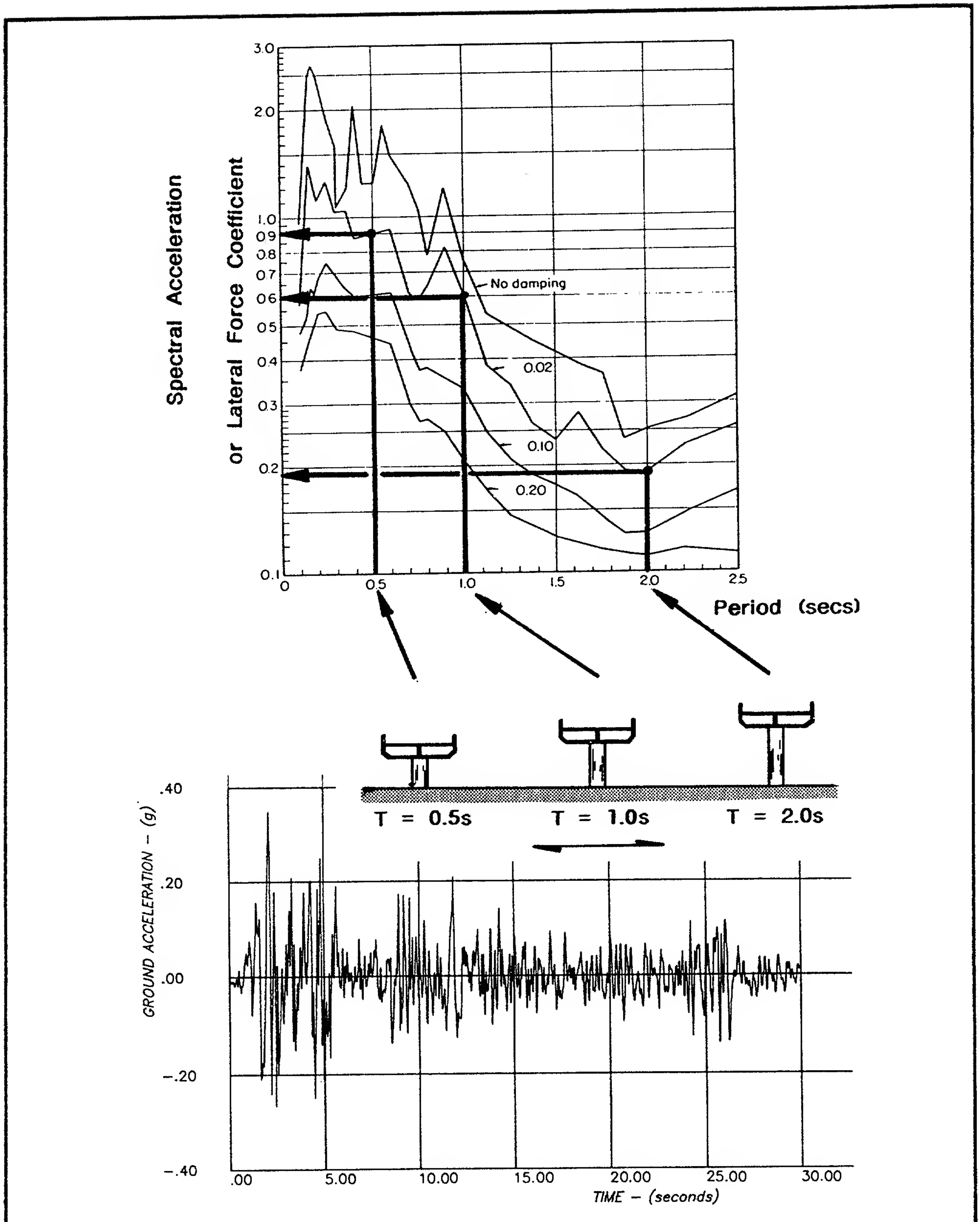


Figure 14: Acceleration Spectra (Lateral Force Coefficients) for the El Centro (1940 NS) Earthquake

Because of the general trend of acceleration spectra to decrease with increasing period, this longer period is beneficial to the structure because less load is now induced. For example, if the period should shift to 1.5 secs., it can be seen from figure 14 that the peak acceleration reduces to about one-quarter (0.25g) of its previous value. The total collapse potential of the structure is therefore reduced, but it will be damaged as a result of the initial, very short period.

Figure 14 also shows the response of two bridges with periods of 1.0 and 2.0 seconds. Peak accelerations are 0.6g and 0.2g respectively (assuming 2 percent damping) showing the general trend for reduced accelerations with increasing period. Because the spectra are comprised of jagged lines and not smooth curves, apparent anomalies are possible in the results from these curves. For example, a bridge with a period of 0.75 secs and 2 percent damping will also experience 0.6g peak acceleration whereas at 0.9 sec the acceleration is higher at 0.8g. This is due to a local "valley" in the 2 percent spectra at just this period. Consequently response spectra for design purposes are smoothed to remove these local discrepancies, so that slight changes in period do not give overly favorable or unduly conservative estimates for acceleration.

A careful study of Duhamel's integral will show that the acceleration, velocity and displacement spectra are not independent for a given earthquake. Rather, they are linked by the natural frequency, p, of the vibrating system.

If S_a, S_v and S_d are defined as the maximum value of acceleration, velocity and displacement, respectively, for a given structure and a given earthquake, it can be shown [reference 17] that:

$$S_a = p^2 S_d$$

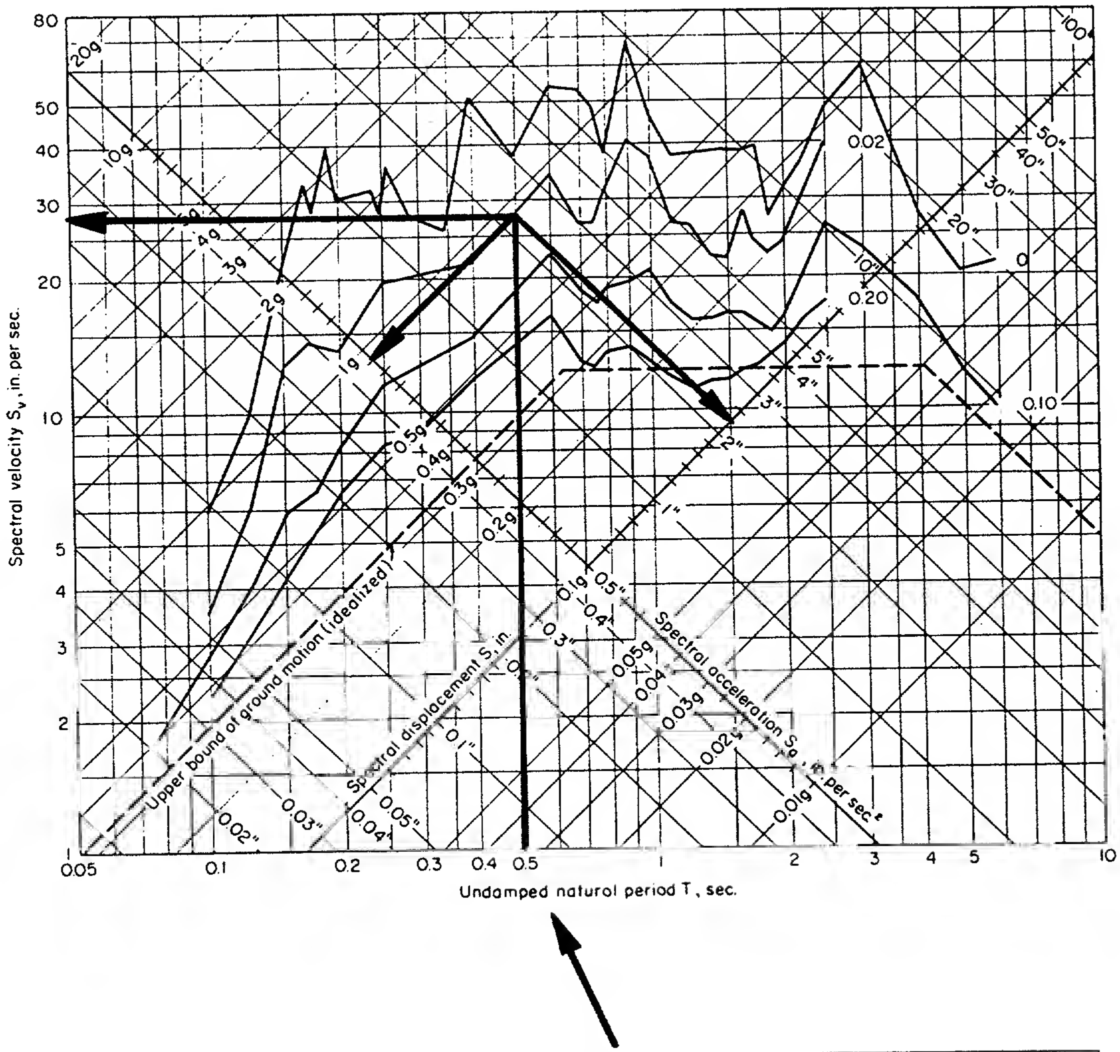
$$\text{and} \quad S_v = pS_d \qquad (10)$$

These three quantities are called **spectral acceleration**, **spectral velocity** and **spectral displacement** respectively and if the frequency term p is replaced by $2\pi/T$, these equations take the alternative form:

$$S_a = 4\pi^2 S_d/T^2 \text{ and } S_v = 2\pi S_d/T. \qquad (11)$$

This interrelationship permits all three spectra to be plotted on the same graph using tripartite axes and logarithmic scales, as shown for the El Centro earthquake in figure 15. It is now possible to also determine the maximum displacement for the example bridge (section 3.1.1) and its maximum velocity, if subjected to the El Centro earthquake. Assuming elastic behavior (no damage) and a period of 0.3 secs., the peak displacement (spectral displacement) is 1 inch. The peak velocity is just over 20 inches/sec. As the period lengthens (because of damage and reductions in stiffness) to 1.5 sec., the displacement increases to more than 5 inches, which confirms the above expectation of excessive deformations (i.e. damage) somewhere in the structure.

Figure 15 also tabulates the reponses of the same family of bridges used in figures 12 and 14. Similar trends are again evident. Damping always reduces response and in some cases quite dramatically. Bridges with longer periods experience lower accelerations and therefore lower seismic forces than those with shorter periods of the same damping ratio. However, one very important consequence for bridges with long periods is the higher displacements that must be accommodated in the piers. As illustrated in figure 15, there can be a sixfold increase in displacement for a

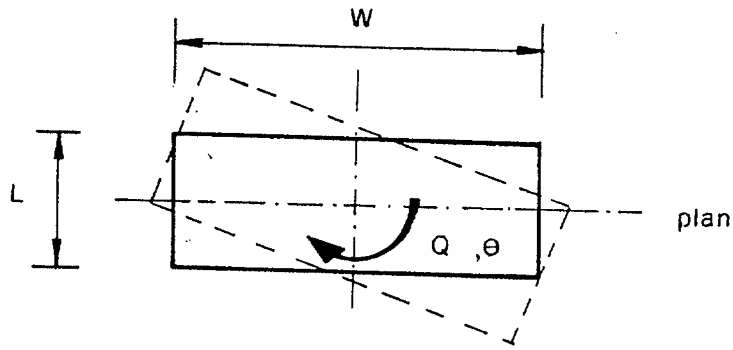


Damping Ratio (n)		0.02			0.10		
Period (T)	seconds	.3	.5	.9	.3	.5	.9
Spectral displacement (S_d)	ins	1.0	2.2	6.0	0.6	1.5	2.9
Spectral velocity (S_v)	ins/sec	20.5	27.5	40.0	12.0	19.0	20.0
Spectral Acceleration (S_a)		1.1g	.9g	.7g	.7g	.6g	.35g

For comparison of performance, the above table summarizes the responses of 6 bridges subject to the same El Centro earthquake. These bridges have periods of 0.3, 0.5 and 0.9 seconds and damping ratios of 2% and 10%.

The results for the 0.5 second period bridge with 2% damping are illustrated in the above Spectra. All remaining results have been read from the same figure in like manner.

Figure 15: Tripartite Plot of the Acceleration, Velocity and Displacement Spectra for the El Centro (1940 NS) Earthquake



ASSUME:

- individual column elements have negligible torsional stiffness
- bridge superstructure acts as a diaphragm rigid in its own plane
- mass of diaphragm is uniformly distributed
- diaphragm interconnects all elements contributing to torsional stiffness
- diaphragm rotates about center of stiffness

NOTATION:

a_x, a_y, a_θ	accelerations in the x, y and θ directions
d_x, d_y	displacements in the x and y directions
F_x, F_y	shear forces in x and y directions
F_{Ix}, F_{Iy}	inertial forces in x and y directions
K_x, K_y	lateral stiffness in x and y direction
K_θ	torsional stiffness
L	total length of bridge
δQ, Q	elemental torque and total torque acting about an axis normal to the bridge deck
δm, M	elemental mass and total (translational) mass of bridge deck
M_θ	rotational mass inertia
T	period of vibration
x, y	cartesian coordinate axes for deck
W	total width of bridge deck
θ	rotation about axis normal to bridge deck

STEP 1 **CALCULATE TORSIONAL STIFFNESS**

Consider one element (e.g., a column) forced by the diaphragm rotation to deflect (d_x, d_y). Then the torque required at the center of rotation is:

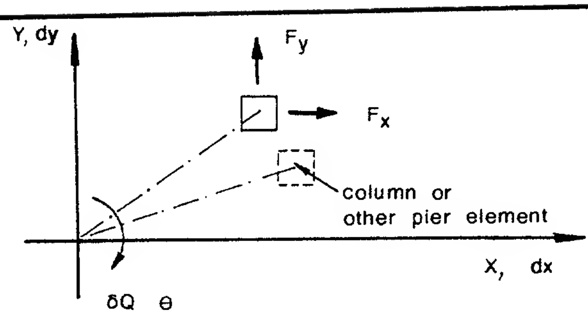
	δQ	$= F_x y - F_y x$
but	F_x	$= K_x d_x$ and $F_y = K_y d_y$
where	K_x	= column lateral stiffness in x direction
	K_y	= column lateral stiffness in y direction
but	d_x	$= y.\theta$ and $d_y = -x\theta$
where	(x,y)	= column coordinates with respect to center of stiffness
and	θ	= diaphragm rotation
substitution gives	δQ	$= [y^2 K_x + x^2 K_y]\theta$

The total torque (Q) is given by the summation over all the columns supporting the diaphragm.

Therefore $$\underline{\underline{K_\theta = Q/\theta = \Sigma\ [y^2 K_x + x^2 K_y]}}$$

This is the required torsional stiffness.

Figure 16: Calculation of Period of Vibration for Rotation about Vertical Axis



STEP 2 **CALCULATE ROTATIONAL MASS INERTIA**

Consider a small element of the diaphragm mass, δm to move (dx,dy) as a result of the diaphragm rotation, θ.

Then inertia forces acting on the element are

$$F_{Ix} = -\delta m a_x \text{ and } F_{Iy} = -\delta m a_y$$

where $$a_x = y.a_\theta \text{ and } a_y = -x.a_\theta$$

Therefore the elemental torque felt at the center of stiffness due to the mass inertial forces is:

$$\delta Q = F_{Ix}y - F_{Iy}x$$
$$= -\delta m(x^2 + y^2)a_\theta$$

and total inertial torque, Q is given by

$$Q = -\int (x^2 + y^2)dm.a_\theta$$

Therefore $$M_\theta = -Q/a_\theta = \int (x^2 + y^2)dm$$

and for a rectangular deck of dimensions (L,W) and total mass M, rotating about its center,

$$\underline{\underline{M_\theta = M\ (L^2 + W^2)\ /\ 12}}$$

STEP 3 **CALCULATE PERIOD OF TORSIONAL VIBRATION**

Torsional period is given by $T = 2\pi \sqrt{M_\theta / K_\theta}$

For bridge in Figure 8: L = 240, W = 30, M = 52.17 $Ksec^2/ft$
and center of stiffness is at center of bridge.

Therefore rotational mass inertia, M_θ = 254,348 $K.ft.sec^2$

Also for same bridge:

abutment lateral stiffness = 2x bent lateral stiffness = 2(9600)K/ft

therefore $$K_\theta = 19{,}200(120)^2 + 0 + 19{,}200(120)^2$$
$$= 552{,}960 \times 10^3 \text{ K ft/radian}$$

Therefore on substitution, $\underline{\underline{T = 0.13 \text{ secs}}}$.

Figure 16: Calculation of Period of Vibration for Rotation about Vertical Axis (continued)

threefold increase in period. Unless the substructures (piers and bearings) are specifically detailed to withstand these displacements elastically, high ductility demands will be imposed which, as noted above, implies structural damage in the piers.

3.5 LIMITATIONS ON SINGLE MODE MODELLING

Whereas the single degree-of-freedom model is extremely valuable, because response spectra are available to predict its behavior, not all bridges and certainly not all buildings can be accurately analyzed in this way.

This is because these bridges can vibrate in other mode shapes besides the fundamental or basic shape and still satisfy equilibrium. Whether these additional modes are important or not is determined by their frequency of vibration and the direction of the inertial forces required to excite each mode. In general, each mode vibrates at a different frequency, and its contribution to the overall response will only be significant if the frequency content of the earthquake includes this particular modal frequency. Also, if the direction of ground shaking is not coincident with the principal direction of the mode, it will not participate in the bridge response. This may not be true for complex bridges with coupled modes, but it is a reasonable assumption to make for regular structures.

Since the frequency content of a typical earthquake is in the range of 0.5 to 20 Hz, modes that fall outside this range may usually be safely ignored. In practice, however, a much narrower frequency range may be used to filter out unnecessary modes. Since the degree of modal participation is affected by the energy content of modes comprising the ground motion, those with the highest energy are the most significant. For typical U.S. earthquakes, the frequency content of these modes appears to be in the range 0.5 to 10 Hz which means that structural modes outside this range may also be ignored.

Irregular or unusual bridges are more likely to have higher modes which will need to be considered. To calculate these, a computer program should be used which will find the shapes and frequencies for all those modes judged to be important by the designer. In the absence of other information to guide the designer, 3 modes for each span might be chosen for preliminary study, up to a maximum of 20-25 modes. Many computer codes now exist which perform these so-called eigen-solutions for elastic space frames, and some are specifically oriented towards bridge analysis. General purposes programs may, however, use different terminology, but the basic purpose is the same. For example, "eigenvector" may be used for mode shape and "eigenvalue" may be used for the square of the natural frequency (p_i) of mode i. These same programs will also determine the importance of each mode and combine the modal responses by one of several approved techniques. More information about these procedures and programs is given in chapters 7 and 10.

As an example of a second mode which will exist in the example bridge of section 3.1.1, consider the torsion mode in which the bridge rotates about an axis normal to the deck. If the lateral stiffness of the abutment structures is not infinite (as assumed earlier) but equal to twice the pier lateral stiffness, the period of this torsional mode is shown in figure 16 to be 0.13 secs. This is about one-half of the transverse translational period (figure 11) and will be important if there is any eccentricity in the as-built bridge between the center of mass and center of translational stiffness. Torsional response is further discussed in chapter 5.

3.6 DUCTILE RESPONSE AND FORCE REDUCTIONS

It was demonstrated in section 3.4.2 that the example bridge in section 3.1.1 would be damaged by the El Centro earthquake. To strengthen the bridge to remain elastic (undamaged) would be uneconomical and difficult to justify for such an infrequent load case. Instead, it is a common design principle to accept some seismic damage in a bridge provided it does not lead to the collapse of the structure. If the structural components, which are expected to resist these extreme forces, are designed to behave in a ductile manner, collapse can be avoided. However, a clear understanding of the implications of nonlinear behavior is required and the demand such response places on the substructures needs to be calculated.

To gain insight into this design approach it is helpful to first consider the behavior of a single degree-of-freedom system responding elastically to an earthquake. It will exhibit a load-deflection relationship of the kind illustrated in figure 17a. Here, b represents the maximum response of the system and the area abc is a measure of the potential energy stored in the system at the time of maximum deflection. As the mass returns to the initial "at rest" position, this energy is converted into kinetic energy.

Now if the column is not strong enough to withstand the full elastic load implied by b, a plastic hinge will develop and the load-deflection curve will be as shown in figure 17b. When the limiting moment capacity is reached in the hinge, deflection proceeds along the path de and e now represents maximum displacement response. The potential energy stored in the system is given by "a-d-e-f" but not all of this energy is recoverable. Only area "e-f-g" is available and is converted to kinetic energy as the mass returns to zero. The remainder "a-d-e-g" is dissipated in plastic deformation (mainly as heat) and is therefore irrecoverable. Hence, although the strength is less and plastic deformation implies a large deflection for negligible additional load, the maximum deflection of an elasto-plastic system is not significantly different to that of a purely elastic one. This is because less energy is being fed back into the system on each return cycle. However, if plastic deformation is accompanied by strength and stiffness deterioration, these deflections will not necessarily be equal or even close. This possibility is discussed later in this section but for the moment equal deflections are assumed and the two previous load-deflection curves are redrawn as in figure 18a.

Now if ductility is expressed in displacement terms, a ductility factor μ may be defined by:

$$\mu = \Delta_u / \Delta_y \tag{12}$$

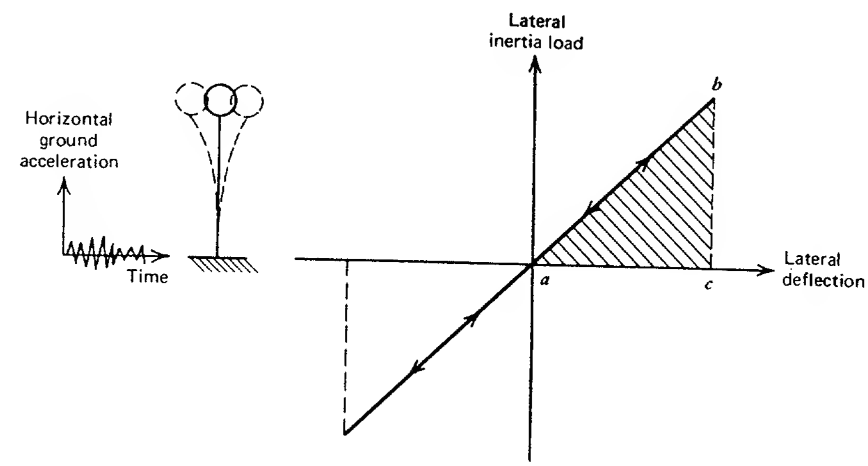
where Δ_u is the maximum lateral deflection at the end of plastic deformation

and Δ_y is the lateral deflection when yield in the column is first reached.

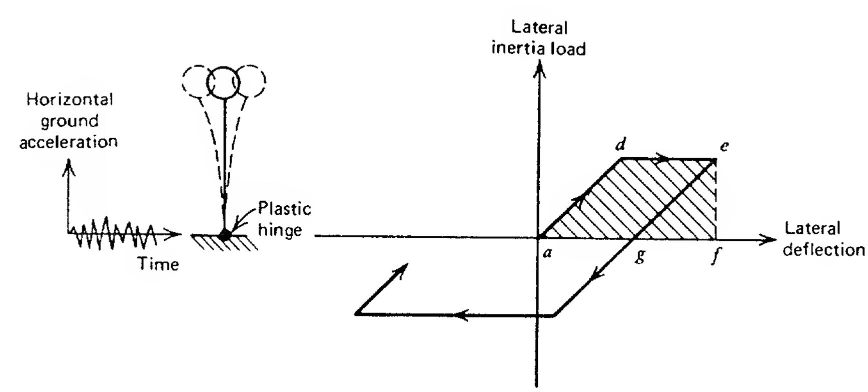
The reduction in force which always accompanies elasto-plastic action can now be expressed in terms of μ. Geometry of similar triangles in figure 18a gives:

$$OB = OA/\mu$$

That is: maximum force for design = maximum force from an elastic response analysis / μ

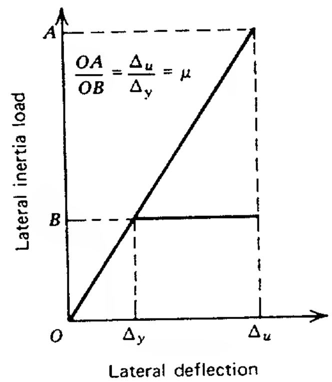


(a) Elastic Response

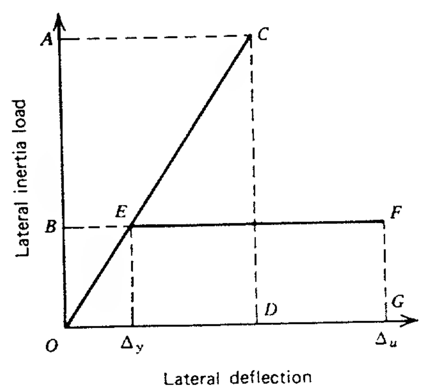


(b) Elasto-plastic Response

Figure 17: Idealized Response of a Single Column Pier (from Reference 20)



(a) Assuming Equal Maximum Deflection



(b) Assuming Equal Work Done

Figure 18: Comparative Response of Elastic and Elasto-plastic Bridge Columns (from Reference 20)

Hence a force reduction factor (R) can be defined and shown to be given by:

$$R = \mu \tag{13}$$

It is well established that columns and piers can be designed to give structure ductility factors in the range of 2 to 5.

This, in turn, means that design forces can be one-half to one-fifth of the elastic response forces, provided the structural components have the capacity for the implied plastic deformations, i.e., provided they can undergo these inelastic deformations **without collapse**.

It is to be remembered that the above development for R is based on the assumption of equal displacements during elastic and inelastic response. However, recent dynamic studies have now shown that the equal-deflection criterion used in figure 18a may not be conservative, especially if the plastic deformation causes a progressive degradation in stiffness from cycle to cycle (as for example in reinforced concrete columns). In these situations an equal energy criterion has been shown to be more appropriate and this is illustrated in figure 18b. Equating the areas "O-C-D" and "O-E-F-G" gives the following expression for R (the force reduction factor):

$$R = \sqrt{2\mu - 1} \tag{14}$$

Considerable judgment is therefore required to obtain suitable reduction factors and both the AASHTO Guide Specifications and the Caltrans criteria make recommendations on appropriate factors to use [references 2 and 4]. Accordingly, R Factors should be selected on a component-by-component basis and take into consideration the importance of the component to seismic performance (e.g. substructure member or a connection detail), structural type (e.g. wall type pier or a multicolumn bent) and material (e.g. steel or concrete). More detailed discussion on ductility and its importance in the seismic design of bridges is given in chapters 4 and 6.

CHAPTER 4 DESIGN PHILOSOPHY

The basic aim of seismic design, as in any engineering design, is to ensure that the resistance of the structure is greater than the loads applied to it. This is complicated in seismic design by the fact that earthquake loads are not deterministic, i.e. they cannot be determined in an explicit manner in the same way that dead loads, vehicle loads and other environmental loads may be computed.

The resistance of a bridge is assessed differently for earthquakes than for other more permanent or frequent loads such as dead and live loads. The magnitude of the most extreme event in a seismically active region will generally be several times as severe as the loads arising from other causes. To design a bridge to remain elastic and undamaged for such infrequent extreme loads is generally uneconomic and in fact not always possible. Therefore the philosophies developed for earthquake resistant design differ from those adopted for other load types.

This chapter presents a seismic design philosophy for bridges and the rationale behind this philosophy. Accordingly, the past performance of bridge structures in earthquakes is reviewed, past and present design criteria are examined and the concept of acceptable damage is introduced. Design concepts for ductile behavior are also reviewed. Finally, a brief outline of seismic isolation for bridges is presented.

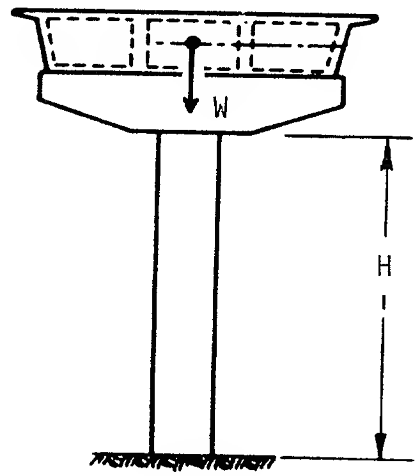
4.1 BASIS FOR BRIDGE SEISMIC DESIGN PHILOSOPHY

For permanent loads (dead loads), or frequently occurring loads (live loads), engineering design is based on elastic principles so that the capacity of the structure is sufficient to resist all loads with a specified margin of safety. The magnitude of earthquake loads is such that this principle would be unrealistic for most bridges. Accordingly, a commonly accepted seismic design philosophy for bridges is as follows:

1. For low to moderate earthquakes, which may be expected to occur several times throughout the life of a bridge, the structure is designed to resist these loads with only minor damage.

2. For severe earthquakes which may occur once in the lifetime of a bridge, some structural damage is accepted but controlled so as to prevent collapse and preserve public safety. Where possible, damage that does occur should be readily detectable and accessible for inspection and, if feasible, repair.

These concepts can be illustrated by means of the simple bridge example shown in figure 19, and the two response spectra given in figure 20. The lower level spectrum in figure 20 is representative of a low to moderate earthquake whereas the higher level spectrum is representative of a more severe event at the same site which is assumed to be in a high seismic zone.



$$T = 2\pi\sqrt{\frac{W}{gK}} \qquad K = \frac{3EI}{H^3}$$

Tributary Weight = W

Note: Lollipop model is used for illustrative purposes only. See restrictions on use in Section 3.1.3.

Figure 19: Simplified Model for Calculation of Transverse Period of Vibration

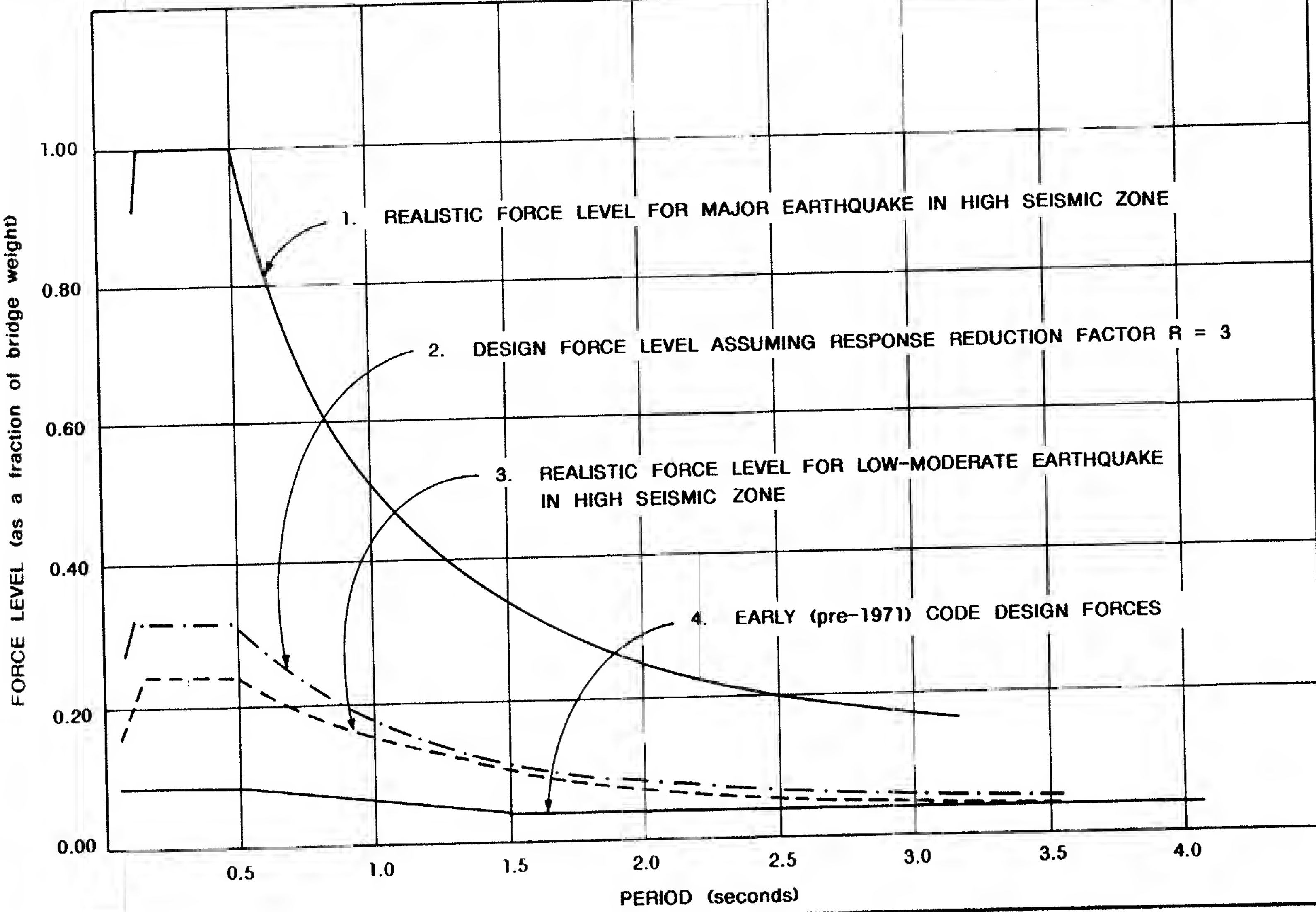


Figure 20: Comparison of Realistic and Design Earthquake Forces

Suppose the period of this bridge, with single column piers, is 0.3 sec.

Then if it is to remain elastic during the severe earthquake for the site, it will need to be designed for a lateral seismic shear force of 1.0W.

However, as stated previously (section 3.4.2), it is uneconomic to design a bridge to remain elastic under such a high lateral load, and a reduced value is used instead. The consequential damage is accepted provided total collapse is prevented and public safety is preserved (see (2) above).

The permitted reduction depends on the ability of the substructures to withstand this damage without collapse. For single column piers of the type illustrated in figure 19, a response reduction factor of 3 is judged appropriate. See, for example, table 5 in section 6.4. Therefore the design force spectrum for the column is one-third of the elastic spectrum as shown in figure 20. The peak design force is now 0.33W. Also shown on figure 20 is an elastic spectrum for a low-to-moderate earthquake for the same site. It falls below the design curve for the column and therefore, the column will not be damaged (but will remain elastic) during this low-to-moderate event--as required by (1) above.

The seismic design philosophies that are currently used in the design of civil engineering structures in the United States are summarized in the following subsections.

A. **Nuclear Power Plants**

Nuclear power plants are designed for two levels of earthquake excitation. The first is called the Operating Basis Earthquake (OBE) and the second and higher level is called the Safe Shutdown Earthquake (SSE). Site specific studies are generally performed to determine the appropriate sizes of these two earthquakes. The design philosophy is such that for the OBE the stress levels in all structures and equipment are of the order of two-thirds the ultimate strength design values. For the SSE the stress levels are permitted to reach the ultimate strength design values. Thus, for nuclear plants, there is a two-level design approach and in both cases, stresses due to seismic loads are less than or equal to the ultimate strength (capacity). Design forces are not permitted to exceed the ultimate strength and therefore no ductility or inelastic demand (damage) is expected in these structures.

B. **Buildings and Bridges**

Seismic design criteria for buildings and bridges throughout the world are generally based on a single level of design earthquake. The design philosophy generally adopted or inherent in building and bridge codes is such that structures are designed to remain elastic and undamaged for small to moderate earthquakes. These are those earthquakes which will occur more than once in the lifetime of the structure. For more severe earthquakes, the intent of these codes is to avoid collapse but to accept that structural damage will occur, some of which may be so severe that repair will not be feasible and demolition, followed by reconstruction, will be necessary. This means that in a severe earthquake, the stresses due to seismic loads will exceed the ultimate strength and inelastic deformations (e.g. plastic hinges) will be imposed. It is however the express intent of these same codes that catastrophic failure of those structural members which are subject to inelastic deformations, be prevented by good detailing practice.

In many of the early bridge and building design codes the lateral design force (F) was expressed as a fraction of the weight (W) of the structure. This fraction was frequently called a design coefficient (C) which had values in the range of 0.03 to 0.15, depending on the period of the bridge and soil conditions of the site. A comparison of this design coefficient against realistic force coefficients for the highest seismic zone of the AASHTO Guide Specification [reference 4] is shown in figure 20. It is evident that the forces given by these coefficients are significantly lower than those that can now be realistically expected.

Following the catastrophic collapse of many bridges on the Golden State Freeway during the 1971 San Fernando earthquake near Los Angeles, the California Department of Transportation (Caltrans) made major revisions to the lateral force coefficient method. The most important change was to use realistic design forces and displacements. For comparative purposes, the forces and displacements in the new Caltrans design criteria [reference 2] are now similar to the higher level response spectra shown in figure 20. This change resulted in significant increases in forces and displacements for the design of all bridge components. For example, the lateral force coefficient for a typical bridge was raised from 0.12 to above 1.0. However, reductions in these forces are permitted according to the importance, function and type of each component. A theoretical basis for these force reduction factors is given in section 3.6. They are further discussed in section 6.4 under the title "Response Modification Factors".

4.2 PAST PERFORMANCE IN EARTHQUAKES

An excellent literature survey [reference 21] chronicles earthquake damage to bridges up until 1971. Reports on bridge damage in later earthquakes are given in references 22, 23 and 24. Damage to bridge structures may occur in the superstructure, the substructure or the approaches. Typical types of damage are discussed below and illustrated, where possible, by means of photographs from past earthquakes. It will be seen that most failures occur from horizontal rather than vertical ground motion.

4.2.1 Superstructure

Loss of support for the girders is the most severe form of superstructure damage, and this may be caused by a lack of continuity in the superstructure, inadequate support lengths for the girders, skew supports which encourage rotation of the superstructure about a vertical axis, or gross movements at the superstructure supports due to some form of soil failure under the piers or abutments. Typical superstructure collapses are shown in figures 21, 22 and 23. Reduction of this type of failure has been the principal aim of the Californian seismic retrofit program in recent years.

4.2.2 Substructure

Substructure damage generally manifests itself in the form of damage to columns, abutments and foundations (piles, footings). Column damage can be caused by flexural failure (figures 24, 25 and 26), shear failure, (figures 27, 28 and 29), and anchorage failure of longitudinal reinforcement (figures 30 and 31). These types of failure modes may also cause collapse of the superstructure by removal of support for the superstructure.

Figure 21: Southern portion San Fernando Road Overhead

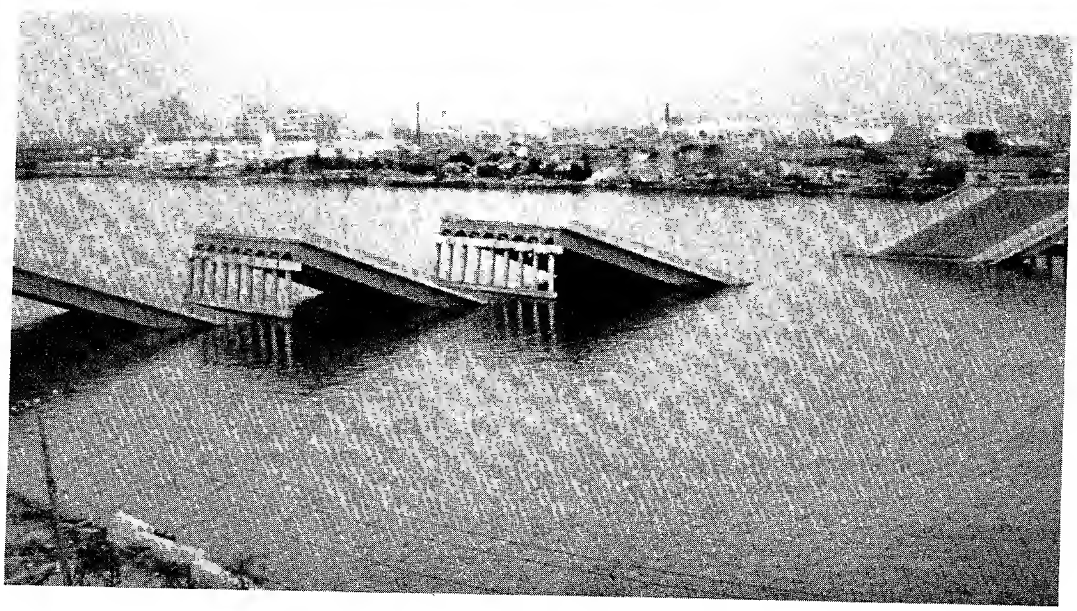


Figure 22: Damage to the Showa bridge

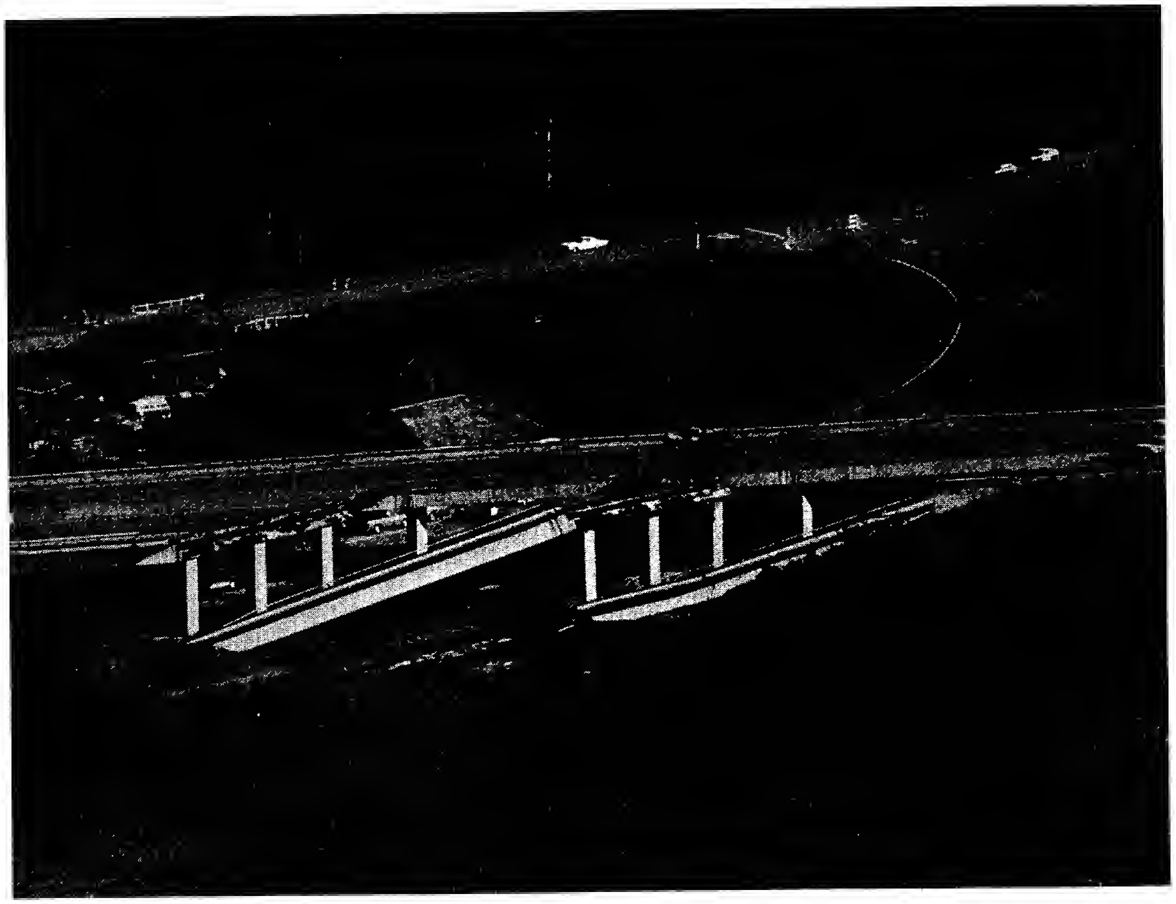


Figure 23: Aerial View of Collapsed Fields Landing Overhead (Photo – Times Standards, Eureka)

Figure 24: Flexural damage in column of San Fernando Road Overhead

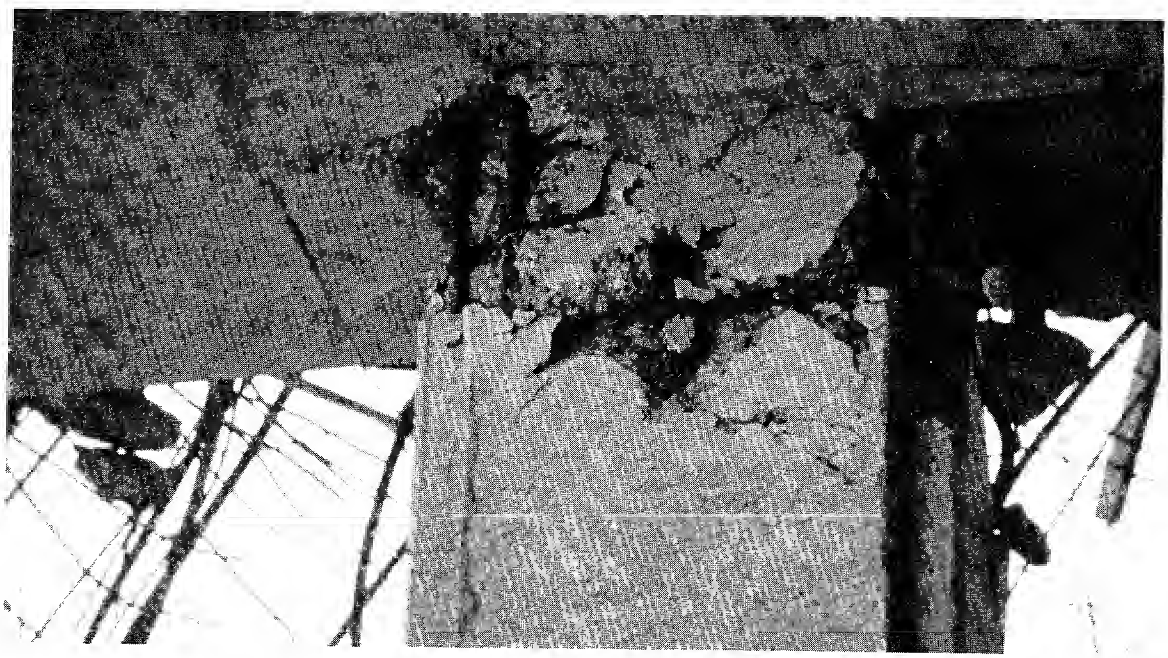


Figure 25: Flexural damage in column of San Fernando Road Overhead

Figure 26: Flexural damage in column of San Fernando Road Overhead

Figure 27: Shear failure in column of San Fernando Road Overhead

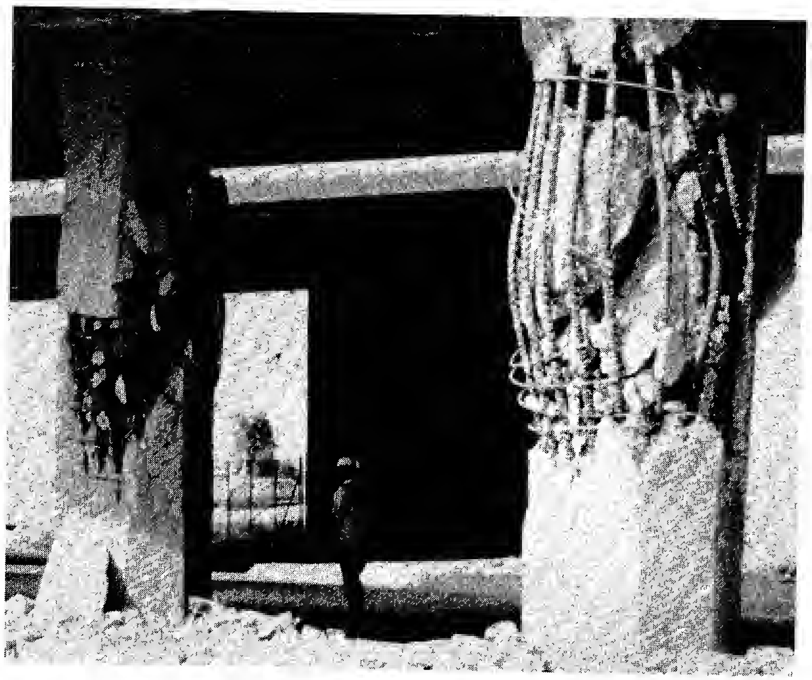


Figure 28: Foothill Boulevard Undercrossing at Foothill Freeway looking in a westerly direction

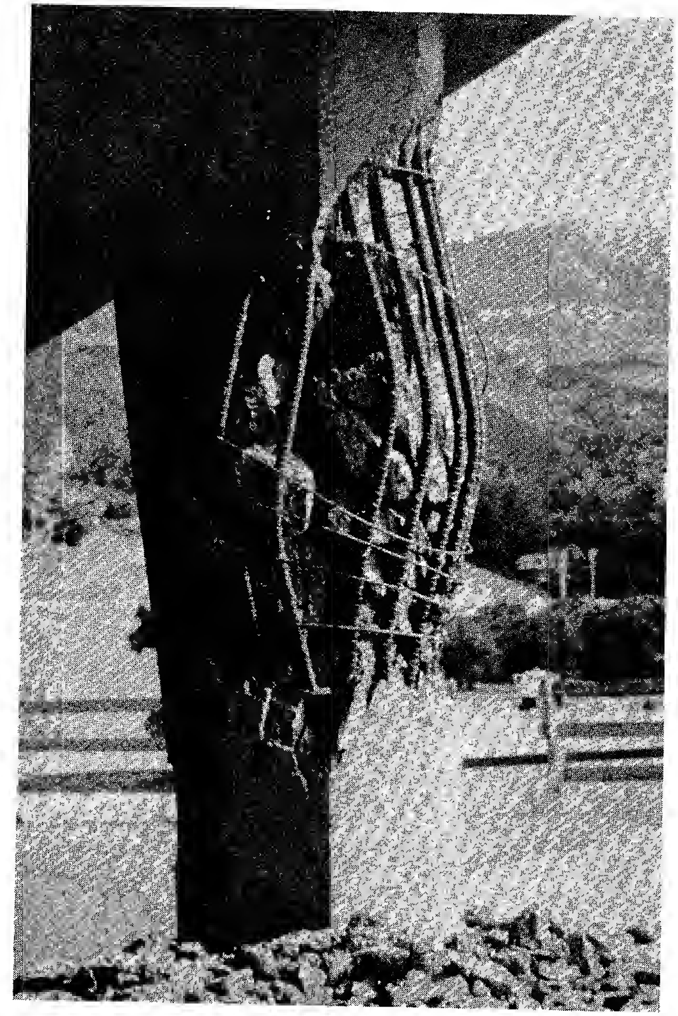


Figure 29: Shear failure of column – Foothill Boulevard Undercrossing at Foothill Freeway

Figure 30: Failure at base of column supported on a single 6-foot diameter cast-in-drilled-hole pile—Golden State Freeway and Foothill Freeway interchange

Figure 31: Failure at base of column supported on spread footing Golden State Freeway and Foothill Freeway interchange

4.2.3 Foundations

Seismic damage, particularly to low bridges, is frequently caused by foundation failures which result from excessive ground deformation and/or loss of stability and bearing capacity of the foundation soils. As a result, substructures often tilt, settle, slide, or even overturn, thus experiencing severe cracking or complete failure. Typical types of failure for spread and piled footings are shown, schematically, in figures 32 and 33 respectively.

4.2.4 Abutments

By virtue of their high lateral stiffness, abutments may attract the largest share of the seismic inertia forces developed in the superstructure. These forces can be very high and may cause severe failures, often of a brittle nature. The interaction of the abutment with the backfill may also cause the wing walls to break loose from the abutments as shown in figure 34. Backfill settlement resulting from compaction is often observed as shown in figures 35 and 36.

4.3 CURRENT BRIDGE DESIGN CRITERIA

Since the 1971 San Fernando earthquake, the Federal Highway Administration has funded numerous research projects to improve the seismic design of bridges. These culminated in a contract to the Applied Technology Council of California to compile a new set of Design Guidelines based on the results of this research. Published in 1981, the ATC-6 Seismic Design Guidelines for Highway Bridges [reference 3] were adopted by AASHTO in 1983 as "Guide Specifications for the Seismic Design of Highway Bridges" [reference 4]. These specifications represent the state-of-the-art in seismic design for bridges and are recommended for the design of all new bridges throughout the United States.

Alternately, the criteria in the AASHTO Standard Specifications [reference 1] may be used in the United States. These were also substantially modified following the 1971 San Fernando earthquake and the revisions first appeared as Interim Specifications in 1975.

As noted above in section 4.1(B), the Caltrans criteria [reference 2] for the seismic resistant design of highway bridges were also rewritten post-1971 and in many respects are similar to the AASHTO Guide Specifications [reference 4].

New Zealand and Japanese engineers have also refined and updated their seismic design criteria for highway bridges in the past five years. As a consequence, the seismic design provisions in the New Zealand Ministry of Works Highway Bridge Design Brief [reference 25], and the Japanese Specifications [reference 26], have recently been amended.

Conceptually, the Caltrans, New Zealand and Japanese seismic design approaches all employ a "force design" concept. The Japanese criteria incorporate the highest levels of design forces and therefore rely less on the ductility of the supporting columns.

In the New Zealand criteria, which also accepts the philosophy that it is uneconomic to design a bridge to resist a large earthquake elastically (without damage), bridges

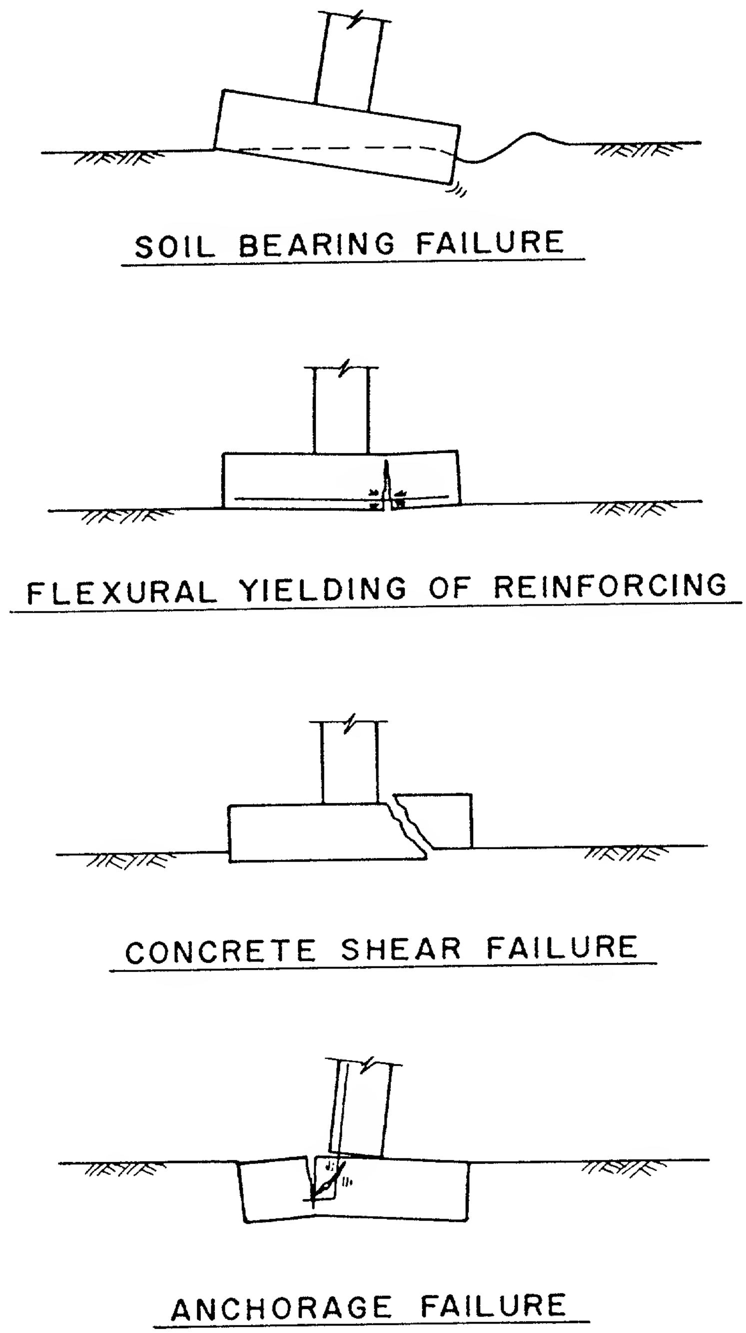


Figure 32: Modes of Failure for Spread Footings

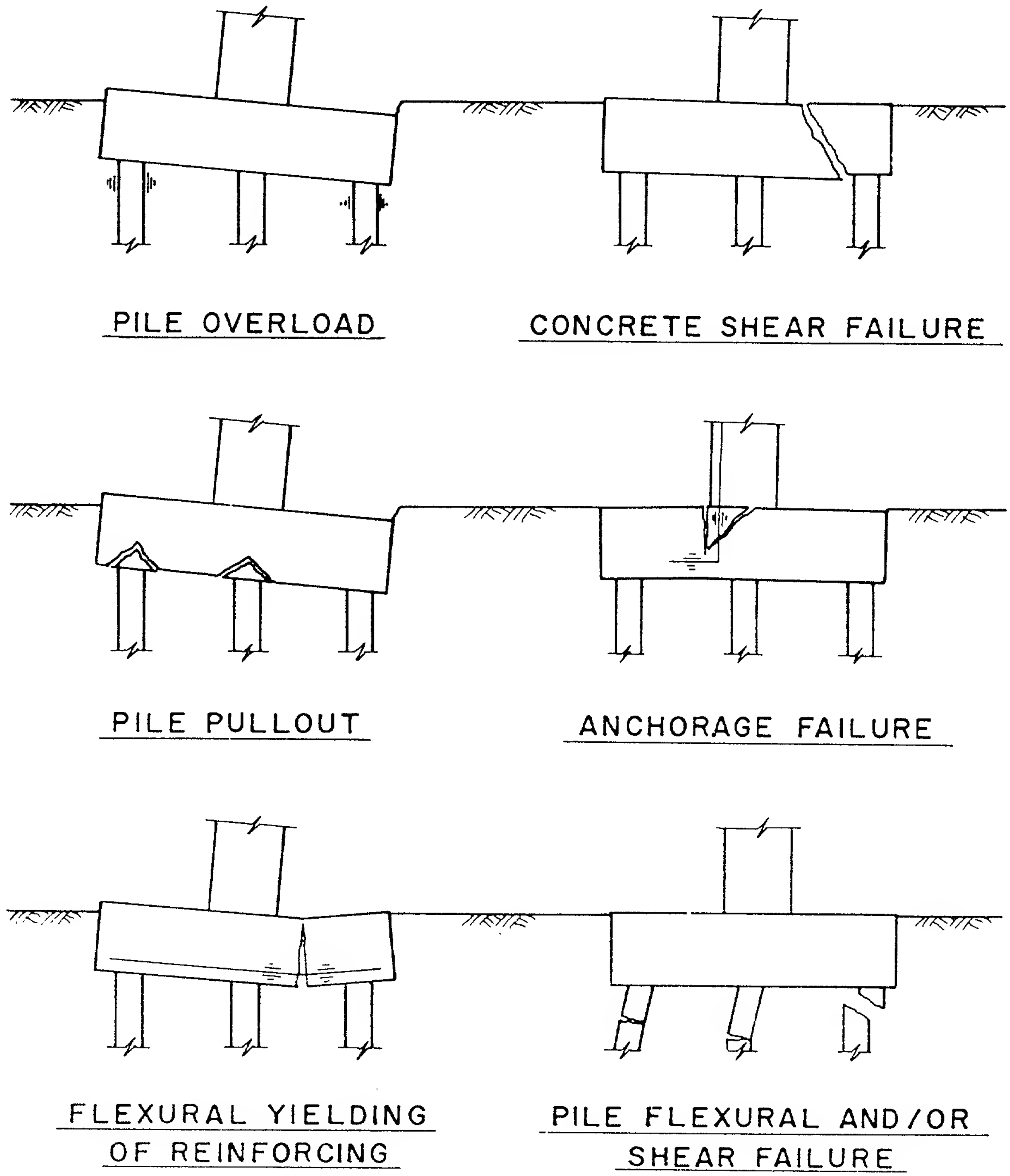


Figure 33: Modes of Failure for Pile Footings

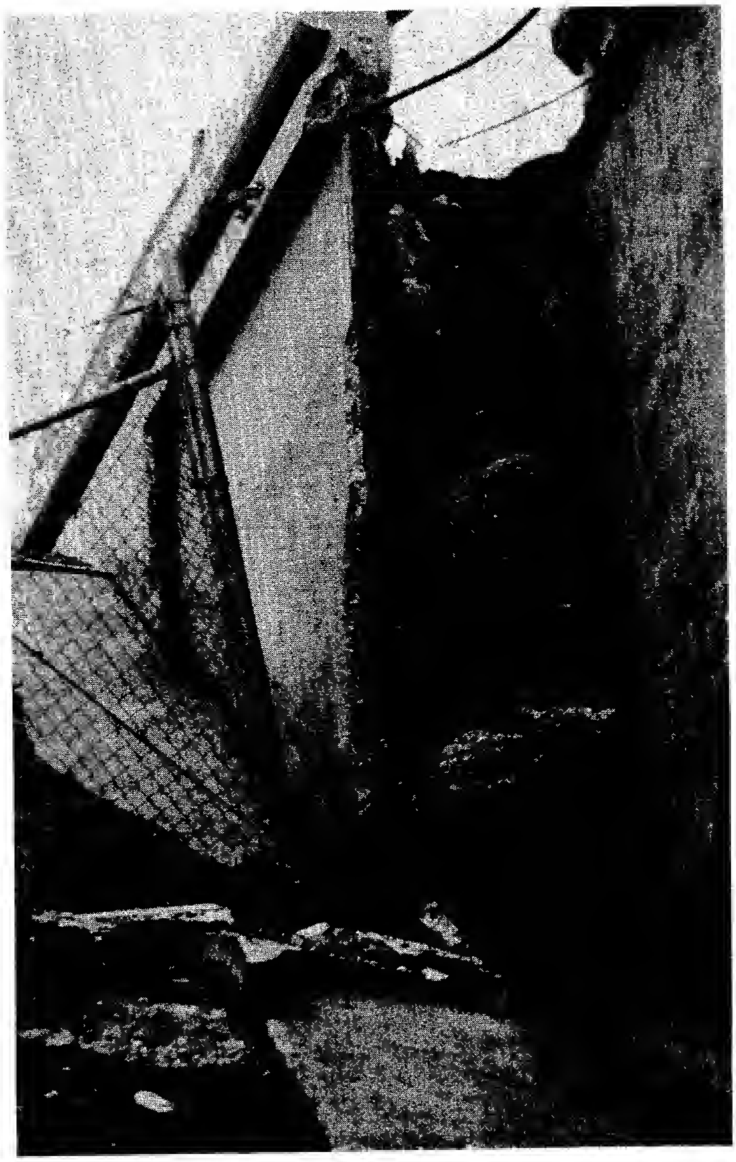


Figure 34: Damaged wing wall of abutment--Roxford Street Undercrossing at Foothill Freeway

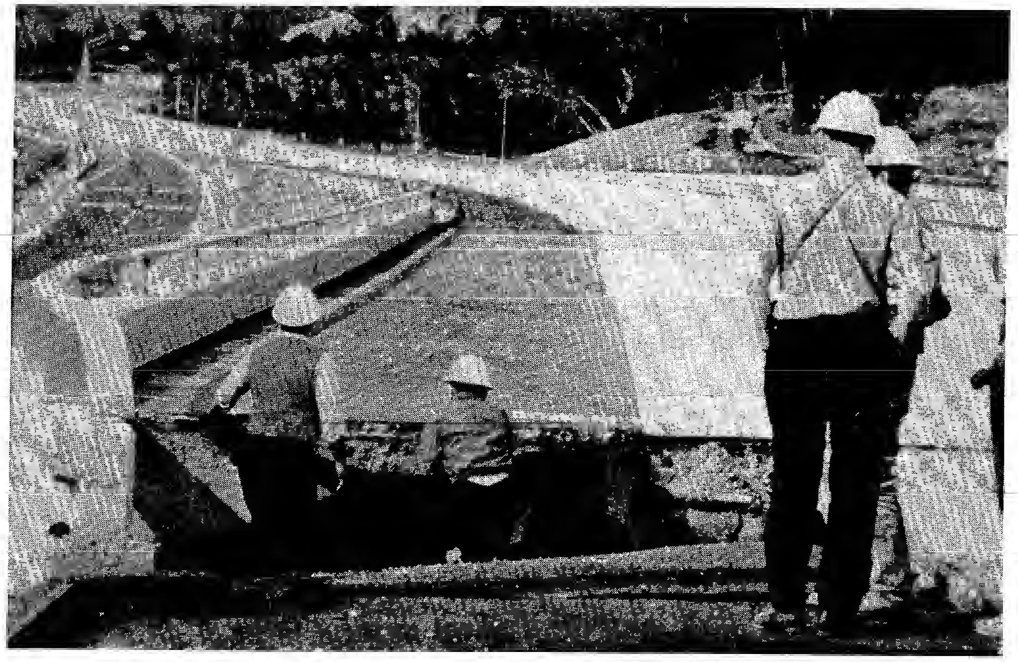


Figure 35: Differential settlement of backfill at abutment Roxford Street Undercrossing at Foothill Boulevard

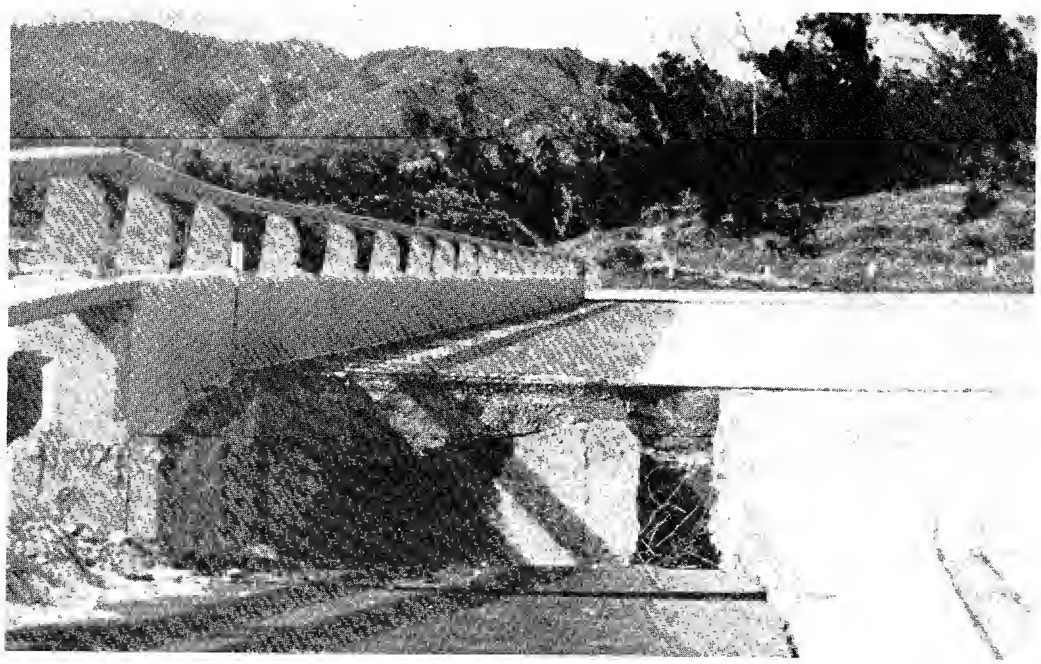


Figure 36: Differential settlement of backfill at abutment Roxford Street Undercrossing at Foothill Boulevard

are designed to resist small-to-moderate earthquakes in the elastic range. Design earthquakes are represented by the upper curve in figure 20, which may be reduced by a displacement ductility factor to determine design force levels. This factor performs a similar function to the reduction factors (R) allowed in other codes. The selection of R depends on the ability of the bridge substructures to withstand inelastic deformation and can range from 2 to 6 according to the judgement of the design engineer. The design philosophy is that columns be capable of resisting the higher forces by inelastic or ductile deformation. Thus flexural plastic hinging in the columns is acceptable but the New Zealand code attempts to prevent significant damage to the foundations and other joints. Consequently, as a second step in the design process, maximum forces resulting from plastic hinging in all columns are determined. These forces are then used for the design of all components connected to the columns including the foundations. Hence, critical elements in the bridge are designed to resist the maximum forces to which they will be subjected by flexural yielding of the columns in a large earthquake.

In the Caltrans approach the member forces are determined from an elastic design response spectrum for a maximum credible earthquake, similar to the upper curve in figure 20. Although there are several spectra in the Caltrans provisions, they are of the order of, or higher than, curve 1 in figure 20. The design forces for each component of the bridge are then obtained by dividing the elastic forces calculated using this curve, by a reduction factor (Z). The Z-factor is 1.0 and 0.8, respectively, for hinge restrainers and shear keys. These components are therefore designed for expected and greater-than-expected (in the case of shear keys) elastic forces resulting from a maximum credible earthquake. Well-confined ductile columns are designed for lower-than-expected forces from an elastic analysis as the reduction factor Z varies from 4 to 8. Thus, in figure 20, the column design forces would be obtained by dividing the upper curve by the Z-factor. This assumes that the columns can deform inelastically when the seismic forces exceed these lower design forces. The end result is similar to the New Zealand approach although the procedures used are quite different.

In the development of the AASHTO Guide Specifications [reference 4], the assessment of many loss-of-span type failures in past earthquakes was attributed in part to relative displacement effects. Relative displacements between adjacent superstructure segments arise from out-of-phase motion of different parts of a bridge, from lateral displacement and/or rotation of the foundations and differential displacements of abutments. Therefore, in the development of the AASHTO Guide Specification the design displacements were considered to be equally important to the design forces. Thus minimum support lengths at abutments, columns and hinge seats were specified; and for bridges in areas of high seismic risk, ties between noncontinuous segments of a bridge are specified. The design philosophy for forces in this AASHTO Specificaton is similar to that of Caltrans. That is, the bridge is analyzed using realistic forces calculated from a realistic design spectrum and the component forces are then modified by dividing these forces by a reduction factor (R)--see section 6.5.

4.4 ECONOMIC CONSIDERATIONS

The design philosophy used for bridges clearly has economic implications. A bridge can be designed such that it will suffer only minor damage in a major earthquake if the upper level curve in figure 20 is used to design all the components elastically. However, the cost increase will be considerable. Thus, in the development of a design

philosophy, clearly stated objectives are required if a compromise between cost and safety is required.

In the development of the AASHTO Guide Specifications [reference 4], both acceptable and unacceptable types of damage were defined. Detailed design and analysis requirements were then developed to achieve these performance criteria as follows.

4.4.1 Acceptable Damage

The only form of acceptable damage in the piers is flexural yielding of the columns. A well designed and detailed steel or reinforced concrete column can be subjected to many cycles of flexural yielding without risk of collapse. Any resulting damage will be visible and repairable and therefore acceptable.

For concrete columns to be repairable, it is most important that the provisions for confinement of the flexural reinforcement be satisfied in the zones where flexural yielding is expected.

Nominal abutment damage may also be acceptable provided adequate seat widths are used to accommodate the larger movements. Such damage might include shear key failure (in the transverse direction) and/or backwall impact (in the longitudinal direction).

4.4.2 Unacceptable Damage

(a) **Loss of Girder Support.** Clearly this is the most unacceptable form of damage. To minimize this potential mode of failure, minimum support lengths for the girders are specified. See sections 7.4.2 and 7.6.8. In addition, the design provisions required for bearings and ties between non-continuous segments are important since a bearing failure may precipitate a loss of girder failure.

(b) **Column Failure.** The two types of reinforced concrete column failures that can lead to a catastrophic collapse are shear failures (figures 27, 28 and 29) and pullout of the longitudinal reinforcement (figures 30 and 31). A capacity design approach (section 4.6) is adopted in the AASHTO Guide Specification to minimize the possibility of a shear failure. Pullout of the longitudinal reinforcement is addressed with detailed design provisions and the requirement to design connections for the maximum expected forces generated from flexural yielding in the columns.

(c) **Foundation Failure.** This can manifest itself in several ways with figures 32 and 33 providing illustrative examples of this type of failure. In addition, any damage that does occur will not be readily visible or easily repairable. As a consequence, the AASHTO Guide Specification minimizes the possibility of these modes of failure occurring. It requires that all foundation structures be designed for the maximum forces that can be transferred by the piers, assuming flexural yield in the piers. These forces may be reduced if it can be shown, for example, that the ultimate soil capacity will be exceeded before these forces reach their maxima. Similarly, local footing uplift is permitted where appropriate.

(d) **Connection Failures.** Connections are extremely important in maintaining the overall integrity of the bridge. Consequently, in the AASHTO Guide Specification particular attention is given to the displacements that occur at moveable supports.

For fixed connections, conservative design forces are specified. In addition, positive horizontal linkage is to be provided between adjacent sections of the superstructure.

(e) **Liquefaction Failure.** Liquefaction of saturated granular foundation soils has been a major source of bridge failures during past earthquakes. For example, during the 1964 Alaska earthquake, 9 bridges suffered complete collapse, and 26 suffered severe deformation or partial collapse. Investigations indicated that liquefaction of foundation soils contributed to much of the damage, with loss of foundation support leading to major displacements of abutments and piers. From the foundation failures documented in the literature, it is clear that the design of bridge foundations in soils susceptible to liquefaction poses difficult problems. Where possible, the best design measure is to avoid deep, loose to medium-dense sand sites where liquefaction risks are high. Where dense or more competent soils are found at shallow depths, stabilization measures such as densification may be be economical. The use of long ductile vertical steel piles to support bridge piers could also be considered. Calculations for lateral resistance would assume zero support from the upper zone of potential liquefaction, and the question of axial buckling would need to be addressed. Overall abutment stability would also require careful evaluation and it may be preferable to use longer spans and to anchor abutments well back from the end of approach fills.

Alternatively, it might be better to take a "calculated risk" for less important bridges in areas susceptible to liquefactiion. In many of these cases, it is not possible to justify the expense of a design that will survive a large earthquake without damage due to liquefaction effects. However, it may be possible to optimize the design so that the cost of repair of earthquake damage does not exceed the cost of additional construction needed to avoid the damage in the first place.

4.5 DUCTILITY DEMAND

It is clear from the discussions of design philosophy that economic seismic design throughout the world in both bridges and buildings is achieved by permitting flexural yielding of the supporting columns. As a consequence, it is imperative that flexural yielding occurs in a controlled and stable manner. Flexural yielding in the column implies deformation beyond the yield capacity of the column (section 3.6). The extent of deformation beyond yield is referred to as the ductility demand on the column. Consequently it is important to understand the definition of ductility and what design parameters impact the ductility capacity of a column.

The following definitions are commonly used to define ductility in a bridge structure, such as that shown in figure 37.

(a) Displacement ductility (structure ductility, figure 38) gives a measure of the extent to which the center of mass of a structure may be displaced beyond the yield displacement.

(b) Curvature ductility (section ductility, figure 39) gives a measure of the extent to which the curvature of a column section may be increased beyond the yield curvature.

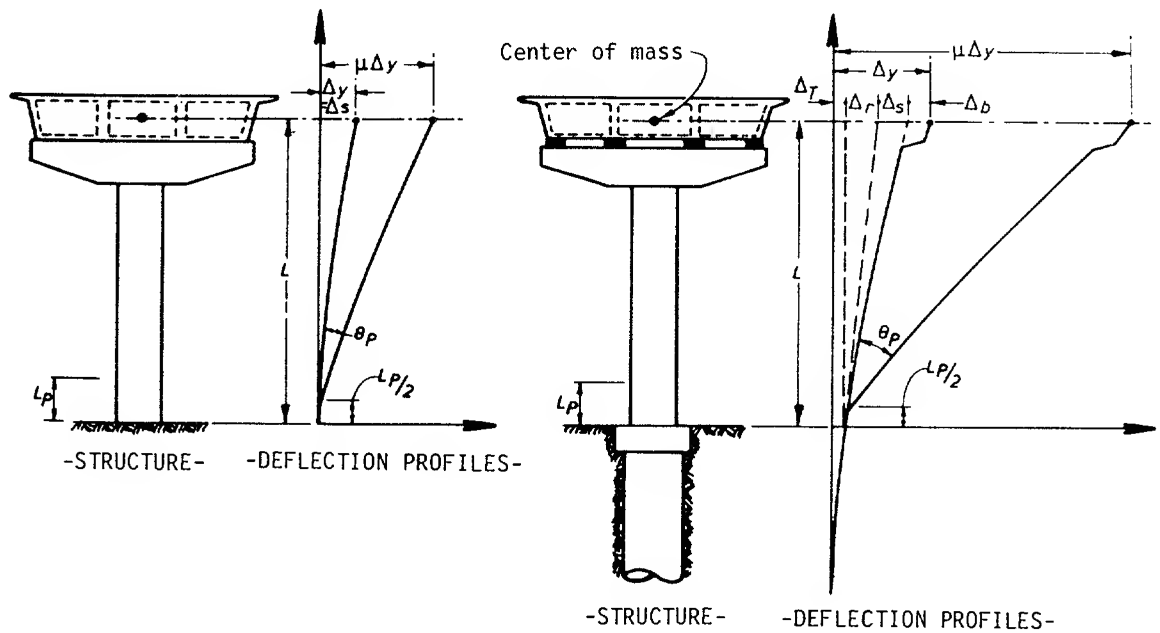


Figure 37: Definition of Elastic and Inelastic Deformations for a Bridge Column

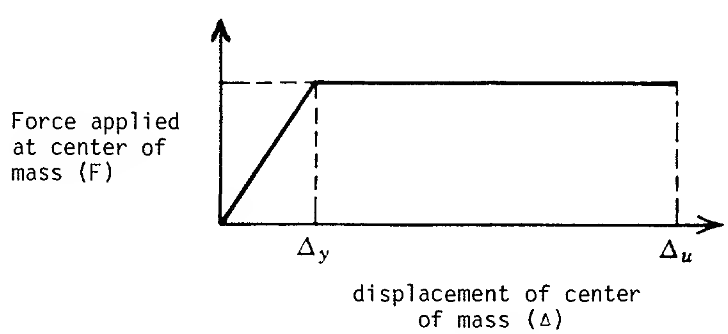


Figure 38: Idealized Displacement (Structure) Ductility

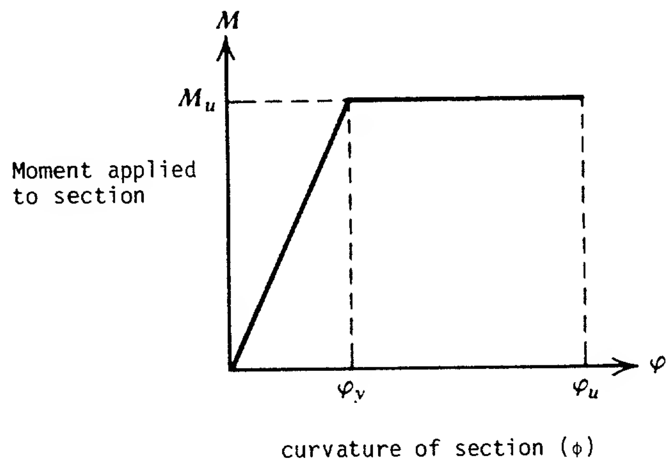


Figure 39: Idealized Curvature (Section) Ductility

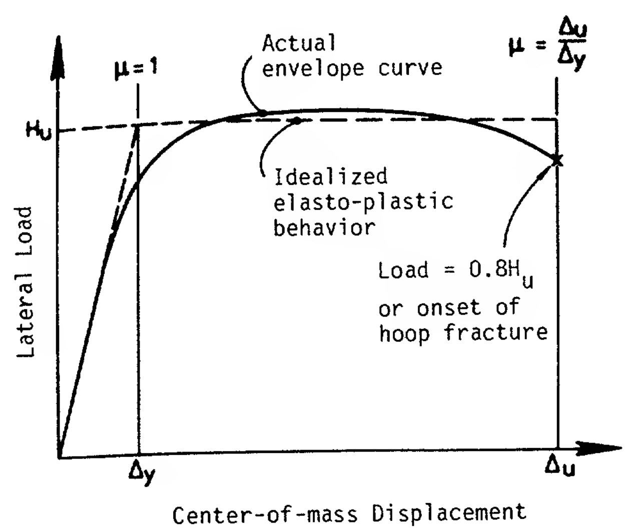


Figure 40: Realistic Force-Displacement Curve Showing Structure Ductility

It should be noted that figures 38 and 39 illustrate idealizations of relationships applying in practice. The actual form of the curves is shown in figure 40. Departure from the elastoplastic idealization occurs because:

(i) Not all of the reinforcing steel in a reinforced concrete section reaches yield at the same time. Bars furthest from the neutral axis yield first. These are followed by the progressive yield of the remaining bars--those closest to the neutral axis being the last to yield.

    A similar situation occurs in structural steel columns when subject to increasing flexural rotations.

(ii) Properties of concrete and steel vary in a nonlinear manner with strain.

For the majority of bridges, where ductility is provided by flexural plastic hinging of the columns, the ductility capacity will be limited by the ultimate displacement Δ_u that can be sustained by the bridge columns without collapse. Definition of Δ_u is somewhat subjective, but a recommended approach for reinforced concrete members is to define Δ_u as the displacement corresponding to either the first fracture of the confining reinforcement in a column plastic hinge (which results in rapid degradation of performance), or to a 20 percent drop in the lateral load capacity after the maximum strength has been reached.

An understanding of the relationship between the structural ductility capacity and the curvature ductility within the plastic hinge zone is fundamental to an assessment of design ductility. The local ductility required at a plastic hinge in a yielding structure may be expressed by the curvature ductility factor Φ_u/Φ_y, where Φ_y is the curvature of the section at first yield, and Φ_u is the maximum imposed curvature.

It can be shown that the required curvature ductility factor Φ_u/Φ_y at the plastic hinge sections will generally be much greater than the required displacement ductility factor for the structure, since once yielding commences, further displacement occurs mainly by rotation at the plastic hinges. The relationship between the curvature ductility demand at the plastic hinges and the displacement ductility demand can be determined by considering the geometry of the deformations of the bridge structure. See, for example, reference 27.

In the AASHTO Guide Specification, there is no requirement to calculate the structural or curvature ductility demand. However, implicit in the requirements are structural, and therefore curvature, ductility demands. The larger the R-factor for columns, the higher the structural ductility demand. For important and very flexible structures, an assessment of the structural ductility may be warranted. In this case, reference 27 provides an excellent summary of the state-of-the-art procedures for performing this type of calculation.

4.6 CAPACITY DESIGN

Capacity design is a design procedure that is used to achieve a desirable hierarchy of the failure modes in a bridge. For example, the brittle and catastrophic modes of failure in columns, such as shear or compression, are much less desirable than a flexural mode of failure. See figures 27, 28 and 29. The intent of a capacity design

procedure is therefore to ensure that failure occurs in a flexural mode before it occurs by shear or compression. To establish a failure sequence in a complex chain of events, it is necessary to know the strength of each link. This knowledge must be based on the most probable strengths of the structural components.

Definitions of various strengths used in capacity design are:

(A) Ideal Strength, S_i

The ideal or nominal strength of a section of a member S_i is obtained from theory predicting the failure behavior of the section based on assumed section geometry and specified material strengths. The ideal strength is that strength to which other strength levels can be conveniently related.

(B) Dependable Strength, S_d

The strength reduction factor (Φ) allows the dependable or reliable strength S_d to be related to the ideal strength by:

$$S_d = \Phi S_i \tag{15}$$

where Φ is always less than 1. This is to allow for material strengths that are less than specified and poorer workmanship and smaller dimensions than assumed for the ideal strength calculation. Each one of these may be within tolerable limits but in combination they will result in undercapacity.

(C) Overstrength, S_o

The overstrength S_o takes into account all the possible factors that may cause a strength increase above the ideal strength. These include a steel strength higher than the specified yield strength plus additional strength due to strain hardening at large deformations; a concrete strength higher than specified, section sizes larger than assumed, and additional reinforcement placed for construction purposes or unaccounted for in calculations. The overstrength can be related to the ideal strength by:

$$S_o = \Phi_o S_i \tag{16}$$

where Φ_o is the overstrength factor and allows for all possible sources of strength increase. It is always greater than 1.

The implementation of the capacity design approach in the AASHTO Guide Specification is to ensure that the columns will have adequate flexural (ductility) capacity and that the brittle shear and compression modes of failure have a low probability of occurrence.

The steps required to achieve this are as follows:

STEP 1 : Analyze the structure under the design earthquake load and reduce the forces so obtained by the appropriate R-factor. Perform the specified combinations with other loads to calculate the required flexural strength M_n at the plastic hinge sections of the columns.

STEP 2 : Design the plastic hinge sections of the columns such that the dependable flexural strength $M_d \geqslant$ required flexural strength M_n from Step 1.

After finding the governing or controlling axial force and moment combination in Step 1, the column dimensions and longitudinal reinforcement (in the case of a reinforced concrete column) are selected so that:

$$M_d = \Phi M_i \geqslant M_n \tag{17}$$

in which
M_d = dependable flexural strength;
Φ = strength reduction factor;
M_i = ideal flexural strength and
M_n = required moment capacity from Step 1.

The value for Φ varies between 0.5 and 0.9 depending on the level of axial force in reinforced concrete columns.

STEP 3 : Calculate the overstrength flexural capacity of the plastic hinges.

The purpose of the overstrength calculation is to determine the actual member forces that may be present when plastic hinges form in the columns or piers. These overstrength flexural capacities are found from the relation:

$$M_o = \Phi_o M_i \tag{18}$$

in which
M_o = overstrength flexural capacity;
Φ_o = overstrength factor, and
M_i = ideal flexural strength of the column as designed.

The value recommended in the AASHTO Guide Specification for Φ_o is 1.25 for structural steel columns and 1.3 for reinforced concrete columns. The shear force in a column is directly proportional to the column end moments.

i.e.
$$F_v = (M_T + M_B)/h \tag{19}$$

where F_v is the shear force
M_T, M_B are moments at the top and bottom of the column respectively
and h is the column height

This force is a maximum when overstrength values for M_T and M_B are used in equation (19). This maximum is the required shear strength for the column, F_{vn}. To prevent a shear failure, the dependable shear strength (F_{vd}) must be greater than or equal to the required shear strength (F_{vn}).

$$\text{i.e. } F_{vd} \geqslant F_{vn} \tag{20}$$

STEP 4 : Design the columns for shear such that the dependable shear strength F_{vd} exceeds the required shear strength, F_{vn}, from Step 3. This means that:

$$F_{vd} = \Phi F_{vi} \geqslant F_{vn} \tag{21}$$

where Φ is the strength reduction factor (for shear) and F_{vi} is the ideal shear strength for the column.

Designing the shear forces in the columns to correspond to the overstrength moments will ensure that a brittle and undesirable shear failure in the columns has a low probability of occurrence. Since the overstrength moments and corresponding shears are also used to design the connections to the columns and the foundations, the probability of failure in these components is therefore also greatly reduced.

4.6.1 Example

1. Suppose after Step 1 in the above procedure the required flexural strength for a column (M_n) is 100 Kft.

2. Now design a column for this moment and after design, calculate its ideal flexural strength (M_i).

Suppose	M_i	$= 120$ Kft
Assume	Φ, the strength reduction factor in flexure is 0.9	
then	M_d, the dependable flexural strength is given by	
	M_d	$= 0.9 * M_i$
		$= 0.9\ (120) = 108$ Kft
check	$M_d \geqslant M_n$ which it is.	

3. Assume an overstrength factor Φ_o of 1.30 and calculate the overstrength flexural strength, M_o as follows:

$$M_o = 1.30 * M_i$$
$$= 1.30\ (120) = 156 \text{ Kft.}$$

Now calculate the required shear strength (F_{vn}) to prevent shear failure in the columns (assume top and bottom column moments to be equal and a column height of 12 ft)

$$F_{vn} = (M_T + M_B)/h$$
$$= (156 + 156)/12$$
$$= 26 \text{ K}$$

4. If the strength reduction factor in shear is taken as 0.85, the shear steel in the column must be proportioned so as to give an ideal shear strength (F_{vi}) such that the dependable shear strength exceeds the required shear strength.

i.e.	$F_{vd} =$	$0.85 * F_{vi} \geqslant F_{vn}$
i.e.		$F_{vi} \geqslant 26/0.85$ K
i.e.		$F_{vi} \geqslant 30.6$ Kips

5. Compare this value for shear strength (30.6 Kips) with that calculated if just the ideal flexural strength were used and no allowance made for flexural overstrength or for the shear strength reduction factor:

Then: $F_v = (120 + 120)/12 = 20$ Kips

If the column were provided with this level of shear strength, it would most probably fail in shear before the flexural hinges were fully developed.

The difference in the above two shear capacities is the ratio of Φ_0/Φ which for the above example is equal to 1.30/0.85 = 1.53.

4.7 SEISMIC ISOLATION DESIGN CONCEPTS

The isolation of structures from the damaging effects of earthquakes is not a new idea. The first patents for base isolation schemes were taken out at the turn of the century, but until very recently, few structures have been built which use these ideas. Early concerns were focused on the fear of uncontrolled displacements at the isolation interface, but these have been largely overcome with the successful development of mechanical energy dissipators (section 9.5.1). When used in combination with a flexible device such as an elastomeric bearing or a sliding plate, an energy dissipator can control the response of an isolated structure by limiting both the displacements and the forces. Interest in base isolation as an effective means of protecting bridges from earthquakes has, therefore, been revived in recent years. To date there are several hundred bridges in New Zealand, Japan, Italy and the United States which use base isolation principles and technology in their seismic design.

The basic intent of seismic isolation is to increase the fundamental period of vibration such that the structure is subject to lower earthquake forces. However, the reduction in force is accompanied by an increase in displacement demand which must be accommodated within the flexible mount. Furthermore, longer period bridges can be lively under service loads. On the other hand, studies have shown that the cost of the isolation hardware can be offset against the savings in the substructures and foundations (because of the reduced forces) and the long term reduction in repair costs for seismic damage.

There are therefore three basic elements in a bridge isolation system, as follows:

- A flexible mounting so that the period of vibration of the bridge is lengthened sufficiently to reduce the force response.
- A damper or energy dissipator so that the relative deflections across the flexible mounting can be limited to a practical design level.
- A means of providing rigidity under low (service) load levels such as wind and braking forces.

Flexibility : An elastomeric bearing is not the only means of introducing flexibility into a structure, but it certainly appears to be the most practical and the one with the widest range of application. The idealized force response with increasing period (flexibility) is shown schematically in the acceleration response curve of figure 41. Reductions in base shear occur as the period of vibration of the structure is lengthened. The extent to which these forces are reduced is primarily dependent on the nature of the earthquake ground motion and the period of the fixed base structure. However, as noted above, the additional flexibility needed to lengthen the period of the structure

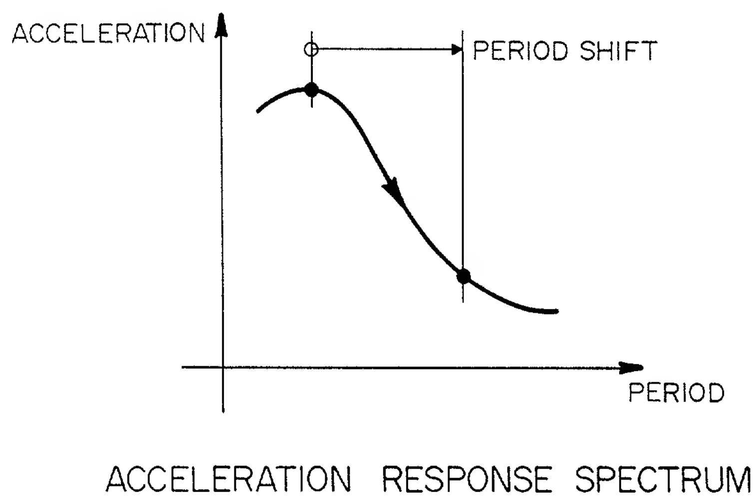


Figure 41: Idealized Force Response Curve

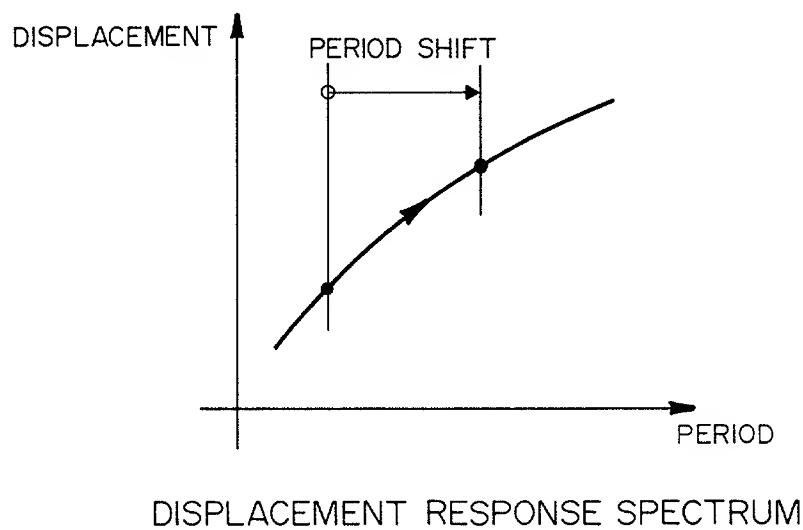


Figure 42: Idealized Displacement Response Curve

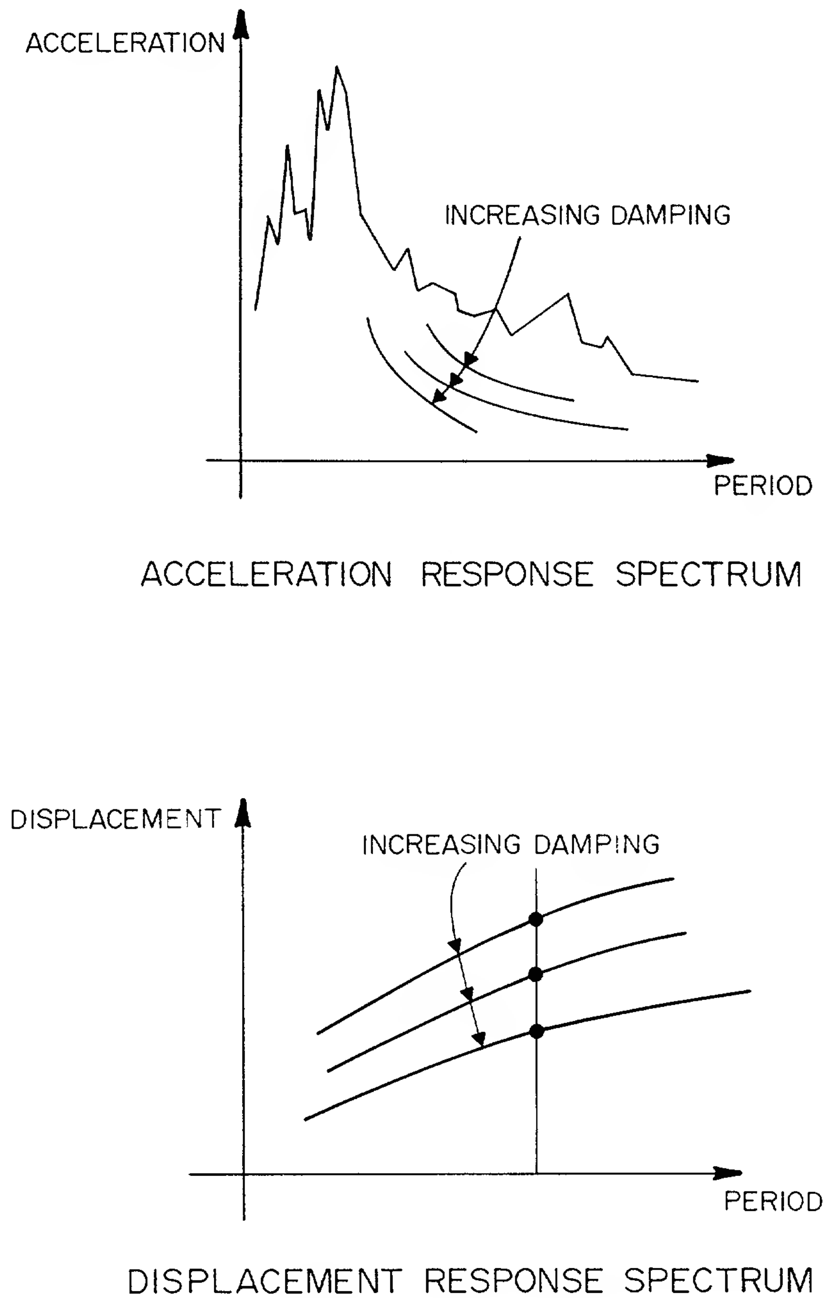


Figure 43: Response Curves for Increasing Damping

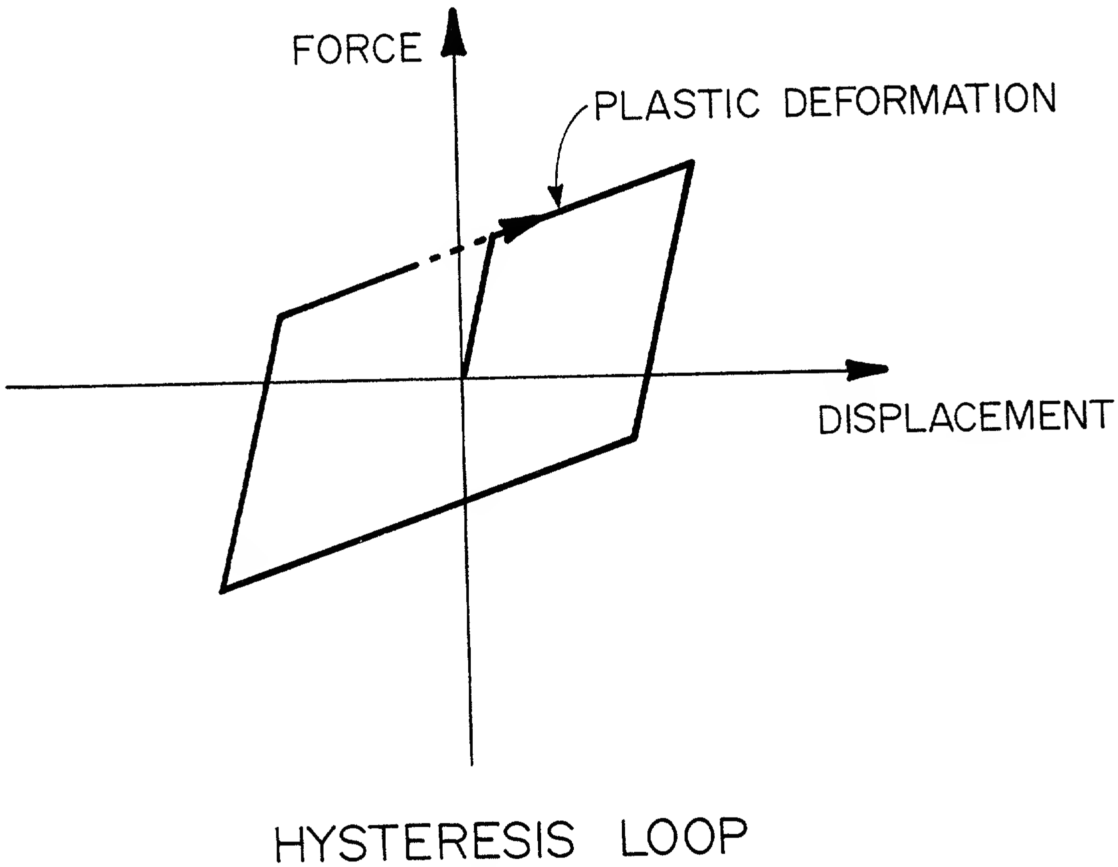


Figure 44: Idealized Hysteresis Loop

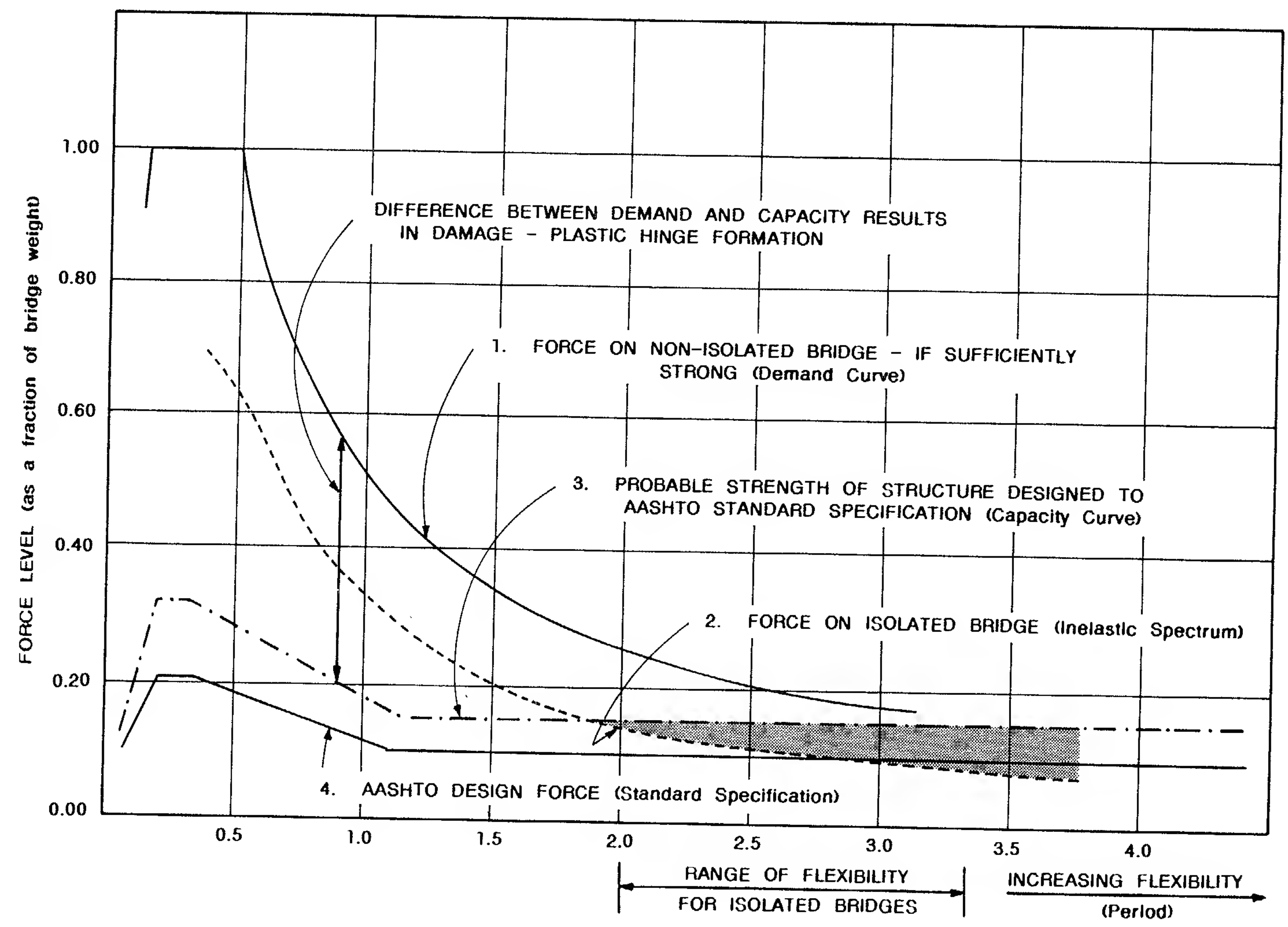


Figure 45: Earthquake Forces

will give rise to large relative displacements across the flexible mount. Figure 42 shows an idealized displacement response curve from which displacements are seen to increase with increasing period (flexibility).

Energy Dissipation : Large relative displacements can be controlled if substantial additional damping is introduced into the structure at the isolation level. This is shown schematically in figure 43. Also shown schematically in this figure is the smoothing effect of higher damping. This effect can also be seen in figures 14 and 15 where 20 percent viscous damping is shown to remove most of the jaggedness inherent in spectra with low damping.

One of the most effective means of providing a substantial level of damping (in excess of 20 percent) is through hysteretic energy dissipation. The term hysteretic refers to the offset between the loading and unloading curves under cyclic loading. Figure 44 shows an idealized force-displacement loop where the enclosed area is a measure of the energy dissipated during one cycle of motion. Mechanical devices which use the plastic deformation of either mild steel or lead to achieve this behavior have been developed (section 9.5.1). Mild steel bars in torsion and cantilevers in flexure have been tested, refined and are now included in several bridge structures. Similarly, lead extrusion devices and lead-rubber (elastomeric) bearings have also been developed and implemented.

Rigidity Under Low Lateral Loads : While lateral flexibility is highly desirable for high seismic loads, it is clearly undesirable to have a structural system which will vibrate perceptibly under frequently occurring loads such as wind loads or braking loads. Mechanical energy dissipators may be used to provide rigidity at these service loads by virtue of their high initial elastic stiffness. Alternately, some base isolation systems use a separate wind restraint device for this purpose--typically a rigid component which is designed to fail at a given level of lateral load.

4.7.1 Design Principles

The seismic design principles for base isolation are best illustrated by figure 45. The solid uppermost line (curve (1)) is the realistic (elastic) ground response spectrum as recommended in the AASHTO Guide Specifications for the highest seismic zone. This is the spectrum that is used to determine actual forces and displacements to which a bridge will be subjected. The lowest solid line (curve 4) is the design curve from the AASHTO Standard Specification. It is seen to be approximately one-fifth of the realistic forces given by the Guide Specification. This reduction, to obtain the design forces, is consistent with an R-factor of 5 for a multicolumn bent (table 5, section 6.4).

Also shown in figure 45 is curve (3), the probable overstrength of a bent designed to the AASHTO Standard Specification. This has been obtained by assuming an overstrength factor of 1.5 (section 4.6). Curve (3) therefore represents the probable capacity of the bent.

The demand on this bent is represented by curve (1) and the difference between demand and capacity results in damage--possibly in the form of plastic hinging in the columns. This difference is highlighted in figure 45 by the arrow and note just above the legend for curve (1).

Now if the bridge is isolated, the actual shear forces that the bridge will be subjected to may be represented by curve (2) (small dashed line). This curve corresponds to the same seismic input as curve (1) but it includes the effect of the substantial level of damping inherent in hysteretic base isolation systems. The period of the isolated bridge will be in the 2.0 to 2.5 second range and it is seen that in this range the overstrength (actual capacity) of the bent exceeds the realistic forces (demand) for the isolated bridge. This region has been shaded in figure 45. There is therefore no inelastic deformation or ductility required of the bent and elastic performance (without damage) is assured.

The benefits of seismic isolation for bridges may be summarized as follows:

- Reduction in the realistic forces to which a bridge will be subjected by a factor of between 5 and 10 (based on curves (1) and (2) of figure 45 and a period shift, due to isolation, of say from 0.4s to 2s.)

- Elimination of the ductility demand and hence damage to the piers.

- Control of the distribution of the seismic forces to the substructure elements with appropriate sizing of the elastomeric bearings.

- Reduction in column design forces by a factor of at least 2 compared to conventional design (based on curves (4) and (2) of figure 45 and a period shift, due to isolation, of say from 0.4s to 2s.)

- Reduction in foundation design forces by a factor greater than 2.5 compared to conventional design (based on the fact that conventional design requires higher design forces for the foundations than for columns).

CHAPTER 5 DESIGN CONCEPTS

This chapter overviews seismic design concepts for bridges and is arranged in three parts.

The first part discusses structural form and highlights those factors influencing seismic performance. The distribution of lateral stiffness and strength determines the load paths through a bridge. The key to a successful seismic design lies in attracting seismic forces to elements able to resist them without collapse. Simplicity, symmetry and integrity are shown to be the basic steps towards this objective. To illustrate these general principles, this section also contains examples of structural form commonly used for resistance to seismic loads. Both good and bad form is illustrated for the purpose of reinforcing basic design concepts.

In the second part, seismic considerations for bridges of unusual geometry and/or type and for those in difficult or hazardous sites are briefly discussed.

The third part of this chapter reviews seismic design concepts for the major components of a bridge. Performance requirements are reviewed for the superstructure, joints and bearings, the substructures, foundations and abutments.

5.1 STRUCTURAL FORM

Seismic performance is determined by structural form, which in turn, is a function of bridge geometry, structural types and member interconnections. Since seismic loads act predominantly in the horizontal plane, the distribution of these loads to the foundation is determined by the integrity of the superstructure and the relative horizontal stiffness of the various substructures supporting the bridge.

Bridges are conventionally designed to optimize performance for vertical loads and often little thought is given to the resistance of horizontal forces--at least in the conceptual stages of the design process. Consequently when seismic performance is checked, it is for an already preconceived structural form. While this approach is understandable in view of the primary function of a bridge structure, it does not always give a seismically optimum bridge. This situation can be avoided if seismic loads, and the appropriate form to resist these loads, are considered from the outset of the design process. Even in regions of low seismicity, where seismic loads may not govern lateral load performance, the adoption of good (seismic) structural form will benefit the overall design. A bridge with good seismic form is a better bridge no matter which lateral load case governs.

Consideration of seismic performance early in the design process is an achievable target since all stages of bridge design are usually performed in the same office. In contrast, building structures are generally conceived by architects and subsequently

given to engineers for design detailing. The reasons for good structural form are then much harder to impress on those responsible for the conceptual design. No matter how competent the engineer, it is considerably more difficult to make bad form respond well in an earthquake. However, for bridge structures the design engineer has the advantage of more direct control over all phases of the design process and good seismic performance should be the end result.

It is the intent of this section to discuss good and bad form with regard to the seismic performance of bridges. Awareness of the principles presented here, during the conceptual design stage, should avoid the evolution of a design with poor seismic details.

5.1.1 Basic Requirements

There are no universally accepted rules for structural form which will guarantee structural survival in an earthquake. Instead, there are general guidelines which, if followed, will greatly improve the chances of satisfactory performance. These are discussed below under the following headings:

A. Simplicity
B. Symmetry
C. Integrity

A. Simplicity

Historical records of earthquake damage repeatedly demonstrate that the simplest structures have the highest survival rate.

Seismic loads are inertial loads and act through the center of mass of each structural component. The transfer of these loads to the ground by the shortest and most direct path will in general assure the best performance. Simple structures have very direct load paths and hence their good performance record.

One of the advantages of a direct load path is that it is usually obvious and can be proportioned and detailed for the expected loads. Collapse or damage, due to load transfer via a weaker and unexpected path, is then much less likely to occur.

Furthermore, more accurate predictions of performance can be made for simple structures, especially those that can be modelled as single-degree-of-freedom systems. Uncertainties associated with higher modes of vibration are eliminated and reliable estimates of forces and displacements can then be predicted for these structures.

By comparison with other structures, the typical highway bridge is a simple structure and can be given a high degree of protection by taking some straightforward precautions. In a bridge, the heaviest component is the superstructure and the transfer of the inertial loads from the superstructure to the ground by the most direct path through the substructures, is the first step towards good structural form.

However simplicity has one serious disadvantage. A simple bridge does not have the same degree of redundancy as a more complex one. There are not as many (if any) alternate load paths, and there may be no fail-safe or back-up system

should failure occur on the primary load path. Consequently, when a simple bridge fails, it does so catastrophically. Simple structures may also concentrate all their ductility demand (energy dissipation requirements) into a few structural members which, in a bridge, are the piers. Therefore, large plastic deformations can be expected in some columns which, if not detailed for this demand, will fail and thus trigger total collapse of the superstructure. It is therefore imperative that attention to detail, particularly joint and plastic hinge details, be given high priority in the design process. This point is discussed again under C. Integrity.

B. **Symmetry**

Symmetry in plan is usually recommended for all structures in order to minimize rotation about a vertical axis and avoid the damaging effects of these so-called torsional rotations. This is also true for bridges where excessive rotations of the superstructure will cause impact and damage to the abutments and impose torsional shear forces on the piers.

Not only is geometrical symmetry important but also symmetry of stiffness. Each girder, pier, abutment and pile contributes to the total lateral stiffness of a bridge structure, but by different amounts. The centroid of this spatial distribution of stiffness is called the center of stiffness (sometimes also called the shear center) and to avoid rotation, it should coincide with the center of mass. The deck will not rotate if the eccentricity between the resultant of the applied force (which acts through the center of mass) and the resultant of the resisting forces (which passes through the center of stiffness) is zero. Symmetry, therefore, requires that the various sources of lateral stiffness in a bridge (i.e. the piers and abutments) be symmetrically located about the center of mass. For bridges with a uniform weight distribution in the superstructure and uniform foundation conditions, this requirement is satisfied by geometric symmetry, i.e. by piers of equal height and size being located symmetrically about the transverse center line of the bridge.

Symmetry is easier to satisfy if there are no sudden changes in stiffness from one substructure to another. If such a change is unavoidable (as from an abutment to a bent), then a similar change should be provided in the corresponding position at the opposite end of the bridge.

One of the consequences of symmetry is that the superstructure deflects in pure translation without rotation. Under these conditions, the load distribution to the substructures is in direct proportion to the individual lateral stiffnesses--those elements with the highest stiffness attracting the highest proportion of the seismic load.

However, if symmetry is not satisfied and torsional rotations are also present, the load distribution is no longer proportional to lateral stiffness and high loads can be imposed on elements of low stiffness and possibly of low strength.

Furthermore, if there is a nonuniform distribution of strength so that one element yields before another, the sudden drop in stiffness for this element may cause a dramatic shift in the location of the center of stiffness. Rotation will occur which may aggravate an already deteriorating structure and may severely damage one or more substructure elements.

C. **Integrity**

This requirement, simply stated, means that the various components of a bridge must remain connected together during an earthquake. These elements and their connections must have sufficient strength to transfer their loads from one to another and to the ground or to those substructure members designed to dissipate these forces by plastic deformation. It also means that where seat-type supports are used, generous seat lengths are provided to avoid loss of support for the girders.

Continuity of the superstructure to distribute in-plane forces to the piers and abutments is clearly important. Bearings and shear keys must have adequate margins of strength to transfer these superstructure loads to the substructures without risk of failure. To be assured that they will perform adequately in these circumstances they should be designed for seismic forces taken directly from the elastic spectra for the site. Substructures must have sufficient strength to transmit these loads to the foundations or be detailed to dissipate significant amounts of energy through plastic deformation (ductility) without collapse. If ductile behavior is expected from a substructure, there should not be any sudden changes of stiffness within the element. Dramatic reductions in stiffness can place very high demands on the displacement capacity of the flexible portions of the element. If not detailed for these high demands, total collapse may result.

Careful detailing is important for a structure's survival. Generous girder seating lengths, conservative bearing details, confining steel in plastic hinge zones and generous rebar anchorage lengths, shear keys and other restraining devices are examples which help ensure a structure's integrity for seismic loads.

5.1.2 Lateral Stiffness

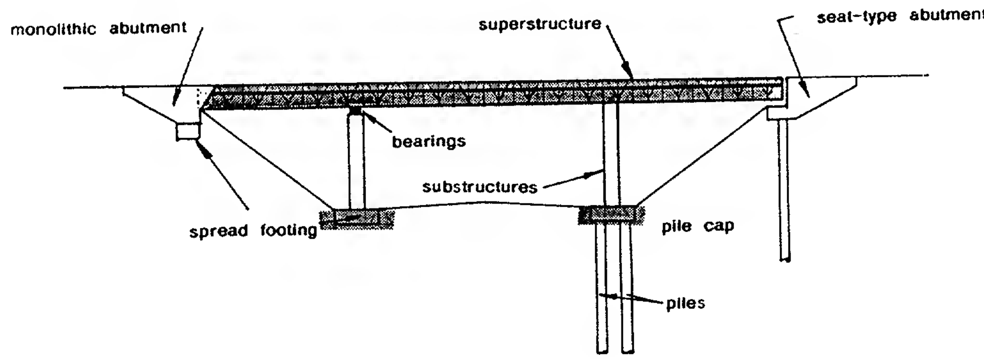
It can be concluded from the above that the distribution of seismic loads from bridge deck to foundation is primarily a function of superstructure integrity and substructure stiffness. However, all elements of a bridge contribute to both the integrity and stiffness and it is now useful to consider the various sources of bridge lateral stiffness. For this purpose the following structural components are identified:

- Superstructure: bridge girders and deck slab.
- Joints, bearings, shear keys, restrainers and other hardware.
- Substructures: single or multi-column bents, wall piers.
- Foundation structures: abutments (both seat-type and monolithic), walls, spread footings, and piled footings.

Figure 46 gives guidance on relative stiffness for each component listed above and also lists those factors affecting lateral stiffness. Useful expressions for computing lateral stiffness are given in figure 47. Most are based on simple beam theory with appropriate modifications for end conditions and shear deformations. A good structural designer's handbook is a useful design aid for this kind of calculation, e.g. reference 28.

5.1.3 Influence of Relative Stiffness on Load Distribution

It was noted in section 5.1.1B that if a bridge deflects in pure translation, the highest loads are attracted into elements of highest stiffness. However, if rotation is present, this general rule no longer holds, and high loads can be generated in elements of



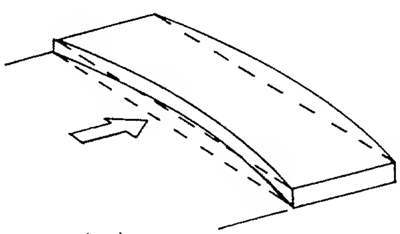
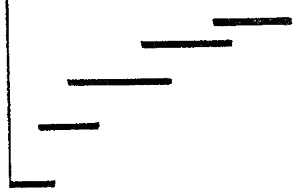
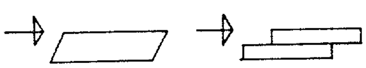
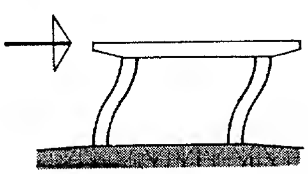
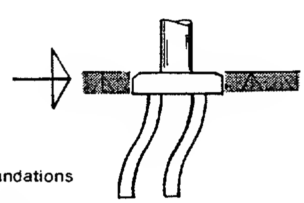
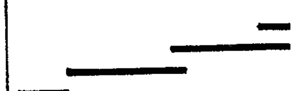
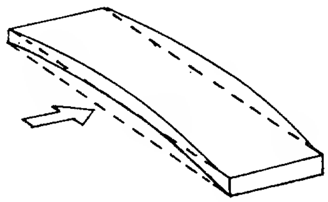
COMPONENT	FACTORS AFFECTING LATERAL STIFFNESS	RELATIVE LATERAL STIFFNESS	
		Component Type	Very Flexible — Very Stiff
 superstructure	total bridge length total bridge width structural type continuity plan geometry	* continuous box girder * continuous T-girder with diaphragms * continuous T-girder without diaphragms * simple spans T-girders with diaphragms * simple spans T-girder without diaphragms	
 bearings	physical dimensions bearing type material properties	* sliding plates with side stops * sliding plates without side stops * elastomeric bearings * shear keys	
 substructure	height section properties structural type end conditions	* single column * multi column * wall pier	
 foundations	effective depth section properties structural type soil conditions	* footings on rock or dense soils * batter piles * vertical piles * spread footings on soft soils	

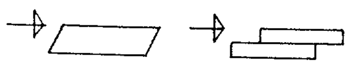
Figure 46: Bridge Components and their Relative Stiffnesses



SUPERSTRUCTURE

(a) point load at midspan — $K = 48\ EI_s/L^3(1 + \alpha)$

(b) uniform load — $K = 384\ EI_s/5L^3(1+0.8\alpha)$

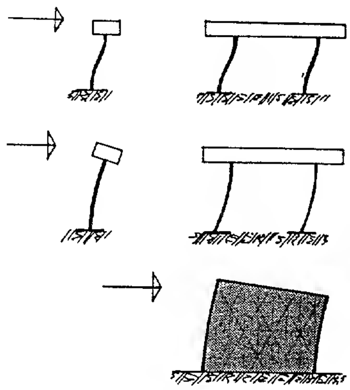


BEARINGS

(a) elastomeric — $K = G_rA_r/T_r$

(b) sliders — very high initially, then zero

(c) pot bearings — very high



PIERS

BENTS

(a) single column: [1,3]

fixed-fixed — $K = 12\ EI_c/h^3$

fixed-pinned — $K = 3\ EI_c/h^3$

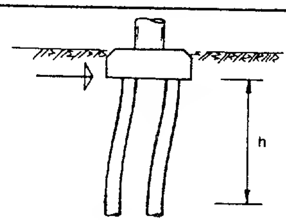
(b) multi-columns: [2,3]

fixed-fixed — $K = 12n\ EI_c/h^3$

fixed-pinned — $K = 3n\ EI_c/h^3$

WALLS

$K = 3\ EI_w/h^3(1 + \beta)$



PILES

as for multicolumn bents
assume h = effective length
to point of fixity

NOTATION

E	elastic modulus
G_r	shear modulus of elastomer
I_s, I_c, I_w	moment of inertia of the superstructure, single column
L	total length of bridge
h	column or wall height
n	number of columns in each bent
A_r	bonded area of elastomer
T_r	total thickness of elastomer
α	$= 12\ EI_s/GA'_sL^2$
β	$= 3\ EI^w/GA'_wL^2$
A'_s, A'_w	effective shear areas for superstructure and wall sections, respectively

Notes

1. If torsional stiffness of superstructure is very high, use fixed-fixed result. If stiffness is very low, use fixed-pinned result. If stiffness is in between, include actual value in calculation for lateral stiffness.

2. If columns neither fixed nor pin ended but framed into bent cap, use a moment distribution procedure to calculate pier stiffness.

3. If pier is piled bent or single column drilled shaft, above expression can be used provided h includes depth to fixity of pile(s).

Figure 47: Approximate Lateral Stiffnesses for Different Bridge Components

low stiffness. This is because the additional deflection in elements some distance away from the center of stiffness, due to rotation, can be very large. Note that such a situation creates a high ductility demand in the adjacent columns.

To illustrate these effects consider the 3-span bridge example in figure 48.

Two cases are presented: The first is symmetric where both abutments and both piers have the same stiffness. For the purpose of discussion each abutment is assumed to be 5 times the stiffness of each pier. In the second case, the bridge is asymmetric with the stiffness of the left-hand abutment reduced to twice the stiffness of a typical pier while the right-hand abutment remains at five times the stiffness.

A. **Symmetric Bridge Example** (figure 48)

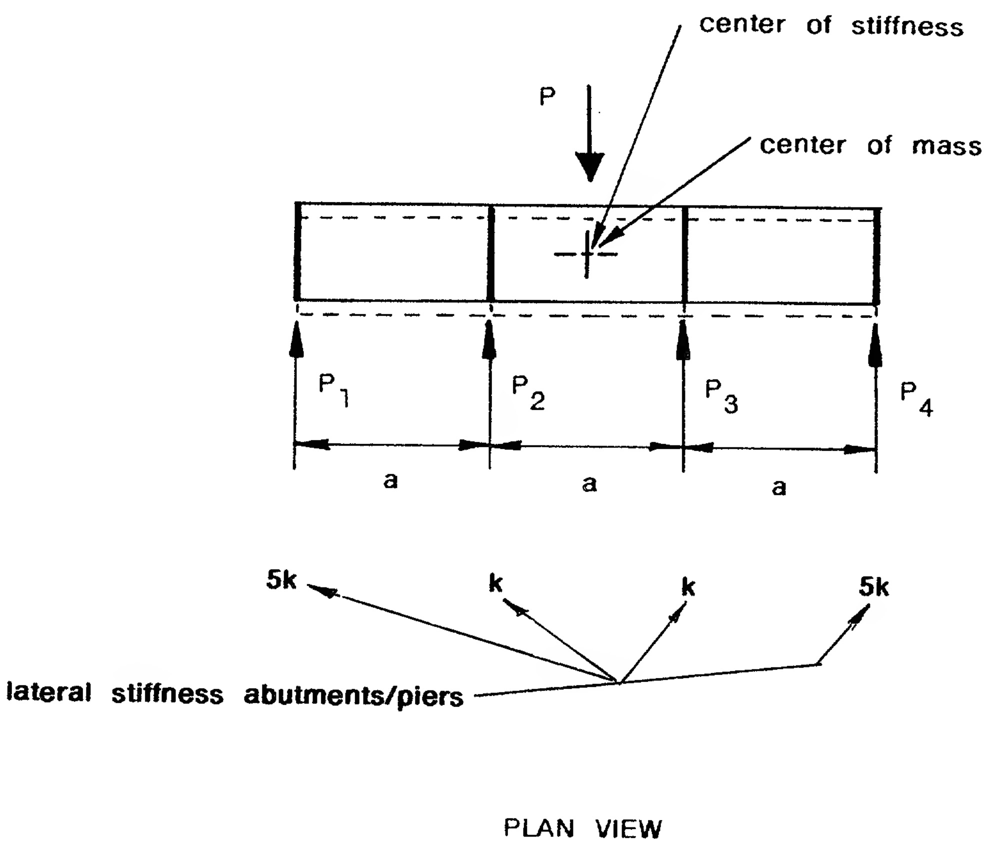
Since the centers of mass and stiffness coincide for this case, the superstructure deflects without rotation; i.e., in pure translation. The seismic load, P, is then distributed in direct proportion to the lateral stiffness of the supporting structures. Thus each abutment attracts about 42 percent of P and each pier attracts about 8 percent of P. Note that these loads are proportional to the stiffness of the elements. Note also that these results are independent of the actual span lengths provided that they are equal to each other. A fundamental assumption has been made to obtain these results and this is that **the superstructure acts as a rigid diaphragm** in its own plane and does not distort or bend under the action of these loads. As noted in figure 46 some superstructures are more rigid than others and the above results must be modified if significant in-plane bending of the superstructure is expected.

B. **Asymmetric Bridge Example** (figure 49)

The reduction in the left-hand abutment stiffness causes the center of stiffness to move towards the right hand abutment. In this example, it is shown to coincide with the location of the right hand pier (RP).

Although most unlikely to occur in practice, it is instructive to first consider what would happen if the center of mass should also move with the center of stiffness and remain coincident with the center of stiffness. Again the superstructure will deflect in pure translation and the load distribution will be in direct proportion to the lateral stiffnesses. It is seen that about 56 percent of P is attracted to the right-hand abutment (the stiffer of the two) while only 22 percent of P is distributed to the left-hand abutment. Each pier then resists only 11 percent of P.

Now consider the more likely situation where the center of mass remains at the geometrical center of the bridge and is thus separated from the center of stiffness by a distance equal to one-half of a span length. The superstructure now deflects in both translation and rotation (torsion) and the total response is the combination of both deformation modes. Figure 49 presents the calculations necessary to find the deflections and forces in the substructures. Table 2 below summarizes these results and compares them against those obtained for the symmetric bridge (figure 48).



ASSUME:

- equal spans, uniform superstructure
- superstructure acts as rigid diaphragm
- abutment stiffness is **five** times **bent** stiffness
- lateral force-deflection relationship for each substructure is given by $P_i = K_i \Delta_i$

  where P_i = load carried by substructure i

  K_i = lateral stiffness of substructure i

  Δ_i = lateral deflection of substructure i

Figure 48: Lateral Load Distribution in a Symmetric Bridge

THEREFORE

- center of mass is at center of bridge
- center of stiffness is at center of bridge
- seismic load, P, acts through center of stiffness
- all substructures deflect same distance laterally, Δ.

EQUILIBRIUM REQUIRES

$$
\begin{aligned}
P &= P_1 + P_2 + P_3 + P_4 \\
&= K_1\Delta + K_2\Delta + K_3\Delta + K_4\Delta \\
&= (K_1 + K_2 + K_3 + K_4)\Delta \\
&= K_T\Delta \text{ where } K_T = \text{sum of lateral stiffnesses}
\end{aligned}
$$

therefore $\Delta = P/K_T$

and $P_1 = (K_1/K_T)P,\ P_2 = (K_2/K_T)P$

$P_3 = (K_3/K_T)P,\ P_4 = (K_4/K_T)P$

let $K_2 = K_3 = K$

then $K_1 = K_4 = 5K$

and $K_T = 12K$

substitution gives $P_1 = P_4 = (5/12)P = 0.417P$

and $P_2 = P_3 = (1/12)P = 0.083P$

also $\Delta = 0.083P/K$

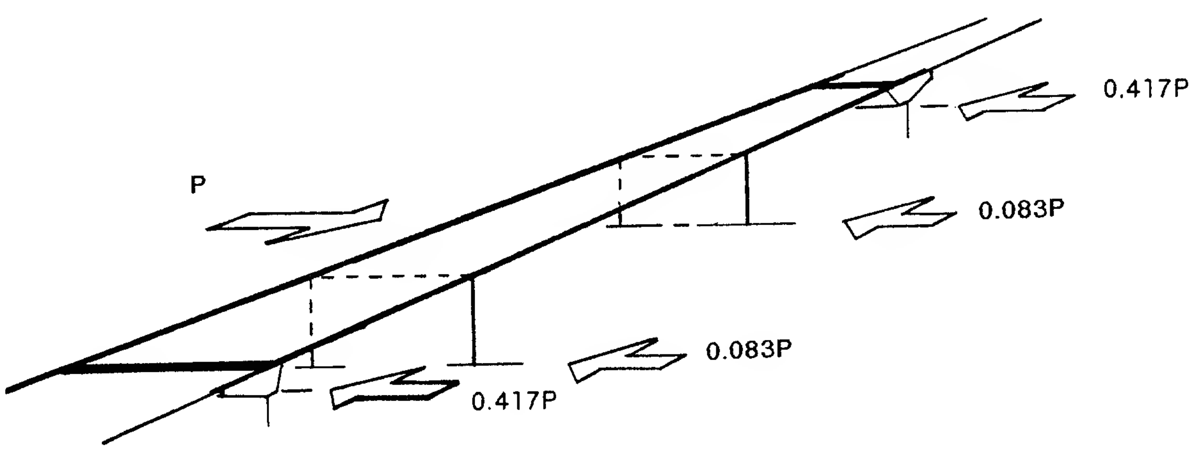
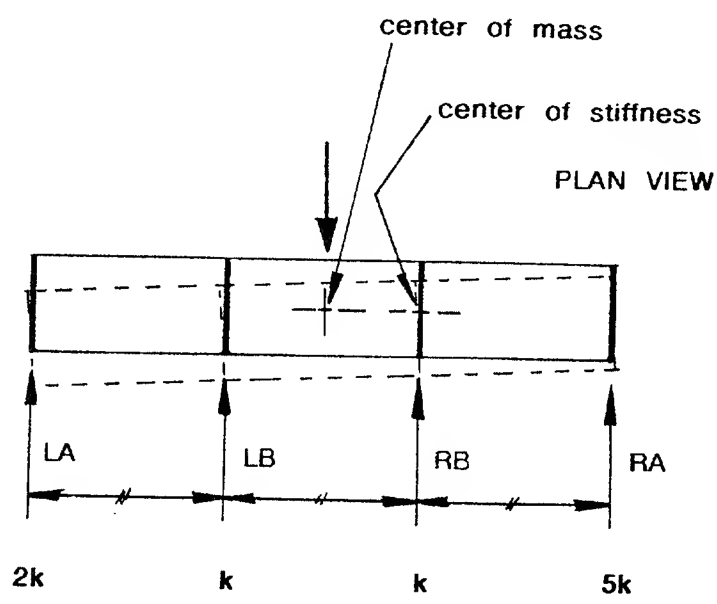


Figure 48: Lateral Load Distribution in a Symmetric Bridge (continued)



ASSUME:

- equal spans, uniform superstructure
- superstructure acts as rigid diaphragm
- left-hand abutment stiffness is **twice** bent stiffness
- right-hand abutment stiffness is **five** times bent stiffness

SINCE stiffness distribution is asymmetrical, locate center of stiffness using centroid of area technique

choose origin at LH abutment

then	$K_T(X) = 2K(0) + 1K(a) + 1K(2a) + 5K(3a)$
where	X = distance from LH abutment to centroid
and	K_T = total lateral stiffness = 9K
substitution gives	$X = 18Ka/9K = 2a$
i.e.	center of stiffness is coincident with RH bent

<u>CASE 1</u> NO TORSION

ASSUME external load, P, passes through center of stiffness. Then calculations for load and deflection are as for symmetric case and can be tabulated as follows:

Abutment/Bent	Lateral Stiffness, K_i	Load, P_i	Deflection Δ_i
LA	2K	.2222P	.1111P/K
LB	1K	.1111P	.1111P/K
RB	1K	.1111P	.1111P/K
RA	5K	.5555P	.1111P/K
TOTALS	9K	.9999P	

Figure 49: Lateral Load Distribution in an Asymmetric Bridge

CASE 2 TORSION

Assume external load, P, passes through center of mass (on bridge center line)
Load is now eccentric to center of stiffness by distance a/2
Eccentric load is equivalent to load P acting through center of stiffness and
Twisting moment T = Pa/2 acting about vertical axis through center of stiffness
Lateral load (P) causes pure translation as in Case 1 above
Twisting moment (T) causes torsional rotation about center of stiffness

Response is the direct addition of both effects

Torsional rotation is given by $\theta = T/K^*$
where $K^* = \sum x_i^2 K_i$ and represents the torsional stiffness of the supporting substructures
Displacements due to torsion are given by $\Delta_{torsion} = x_i\theta$
Calculations can then be tabulated as follows:

Abutment/ Bent	1 x_i	K_i	x_iK_i	$x_i^2K_i$
LA	-2a	2K	-4ak	$8a^2K$
LB	-a	K	-ak	a^2K
RB	0	K	0	0
RA	a	5K	5ak	$5a^2K$
TOTALS		9K		$14a^2K$

Abutment/ Bent	2 Δ_{trans}	3 Δ_{tors}	4 Δ_{total}	5 P_i
LA	.1111P/K	.0714P/K	.1825P/K	.3651P
LB	.1111P/K	.0357P/K	.1468P/K	.1468P
RB	.1111P/K	0	.1111P/K	.1111P
RA	.1111P/K	-.0357P/K	.0754P/K	.3770P
TOTALS				1.0000P

Notes:

1 origin for x coordinates is at center of stiffness (positive to the righ
2 results for pure translation from Case 1 above ($\Delta_{trans} = \Delta_i$)
3 torsional rotation $\theta = T/K^* = Pa/2(14a^2K) = P/28aK$
4 $\Delta_{total} = \Delta_{trans} + \Delta_{torsion}$
5 $P_i = K_i\ \Delta_{total}$

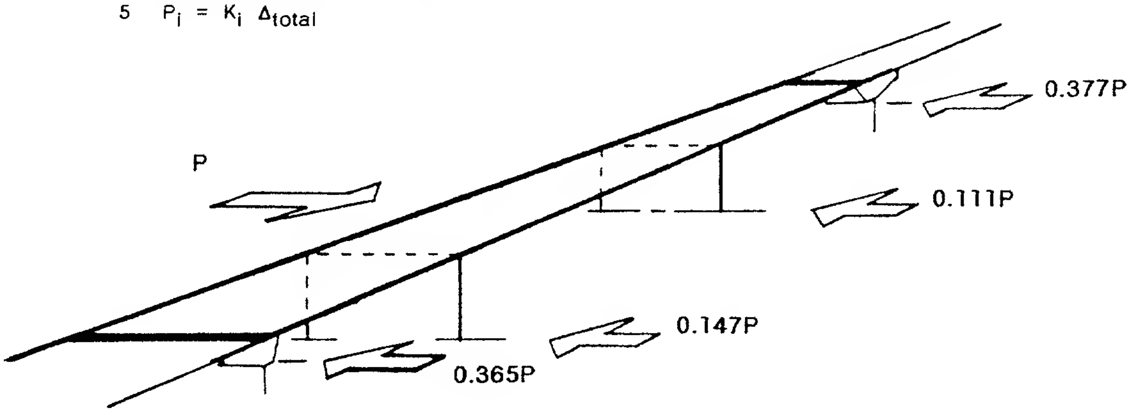


Figure 49: Lateral Load Distribution in an Asymmetric Bridge (continued)

Table 2: Redistribution of Forces due to Changes in Abutment Stiffness

SUBSTRUCTURE	DEFLECTION		FORCES	
	Symmetric Abutments LA = RA	Asymmetric Abutments LA = 0.4 RA	Symmetric Abutments LA = RA	Asymmetric Abutments LA = 0.4 RA
Left Abutment (LA)	.0833 P/K	.1825 P/K	.4167 P	.3651 P
Left Pier (LP)	.0833 P/K	.1468 P/K	.0833 P	.1468 P
Right Pier (RP)	.0833 P/K	.1111 P/K	.0833 P	.1111 P
Right Abutment (RA)	.0833 P/K	.0754 P/K	.4167 P	.3770 P

It is seen that reducing the left hand abutment stiffness causes an increase in its deflection by more than a factor of two. There is a similar increase in the deflection of the left hand pier and corresponding increases in shear forces in both piers (by 75 percent and 25 percent for the left- and right-hand piers respectively).

Despite the fact that the two abutments now have significantly different lateral stiffnesses, they attract almost the same share of the applied load (about 37 percent of P). Since the stiffness ratio is 5:2, the left-hand abutment can therefore be expected to deflect about 2-1/2 times further than the right-hand abutment. Clearly, rotation of the superstructure is playing a significant part in the response of this bridge to the lateral load.

Although every effort might be made at the design stage to keep abutment stiffnesses equal, once an abutment is damaged or the soils yield under an abutment during an earthquake, reductions in stiffness of this order are possible. Redistribution of load immediately follows and deck rotation occurs. This aggravates the situation at the already degrading abutment because of the higher deflections which are now imposed and it also places an additional ductility demand on the adjacent pier.

Again, this example is based on a simple model to illustrate trends in bridge response. The most important simplification is the assumption of rigid body action in the superstructure, i.e. no flexure in its own plane. For certain deck types this assumption will be inaccurate but the principles illustrated above will generally be true. As the superstructure becomes more flexible, the load distribution tends to become more uniform. If the superstructure is perfectly flexible the bridge separates into a number of independent structures and the substructure loads are proportional to the tributary lengths of the superstructure, rather than their lateral stiffnesses. However this latter situation is highly unlikely to occur in typical highway practice but it might be approximated in structures with long slender (narrow) spans as sometimes found in railway bridges.

5.1.4 Load Distribution

Since the load path is determined by lateral stiffness, it follows that the designer can control the distribution of load by adjusting the stiffness of the various supporting

substructures. Changing a single column bent for a wall pier, for example, will make a substantial difference to the transverse loads resisted by that particular substructure.

The impact of unavoidable variations in column height or foundation conditions on lateral stiffness can be softened by making deliberate structural changes to one or more of the substructures and/or abutments. In this way the site constraints can be counterbalanced to some extent.

In both new and retrofit work, the careful adjustment of substructure stiffness can direct seismic loads away from weak elements and foundations and attract them into components more able to resist these loads.

However, there is a limit to the extent that altering the pier section properties can help in this regard. In many situations it is a better strategy to introduce elastomeric bearings between the superstructure and selected substructures. The inherent flexibility of these devices can be used to advantage to achieve a more uniform load distribution or direct load to the desired substructures. The lateral stiffness of these bearings can vary over a wide range, from near zero to almost rigid, by changing the thickness of the elastomer. Control of load distribution is then feasible, despite widely varying substructure and/or foundation properties. Figure 50 illustrates the effectiveness of these bearings in mitigating the impact of large variations in stiffness between piers of different height. In this example, one column is half the height of the other and it is therefore eight times stiffer. This substantial difference in stiffness can be eliminated by introducing elastomeric bearings at the top of one or both columns. As shown in the figure, a pair of 30 inch square elastomeric bearings, each with 2-1/2 inches of rubber and placed on top of the shorter column will balance the lateral stiffnesses almost exactly. Other combinations of bearings and piers are also possible as illustrated in figure 50.

When using elastomeric bearings for this purpose, special care should be taken to prevent the occurrence of undesirable vertical motions especially under vehicular and pedestrian traffic. Elastomeric bearings can, however, be designed to be almost rigid in the vertical direction without seriously affecting their shear stiffness. In these circumstances, amplification of vertical motions through the bearings should not be significant.

Elastomeric bearings are not the only way of improving the stiffness distribution. For example, the base of the shorter columns could be pinned to give a four-fold reduction in stiffness. However, this is not always feasible for single column piers because of the potential for instability in the transverse direction. It is however a practical alternative for multicolumn bents. If significant flexibility is attributable to the foundations, a different piling system (e.g. using batter rather than vertical piles) may help balance the overall distribution of stiffness.

5.1.5 Examples of Acceptable Structural Form

Figure 51 presents several examples of bridge structural form which are considered acceptable from the seismic viewpoint. They are based on the principles outlined earlier and on the historical record of bridge performance in past earthquakes.

The cases illustrated range from single to multispan bridges, with and without bearings and joints, and with and without continuity between spans. This family of structural

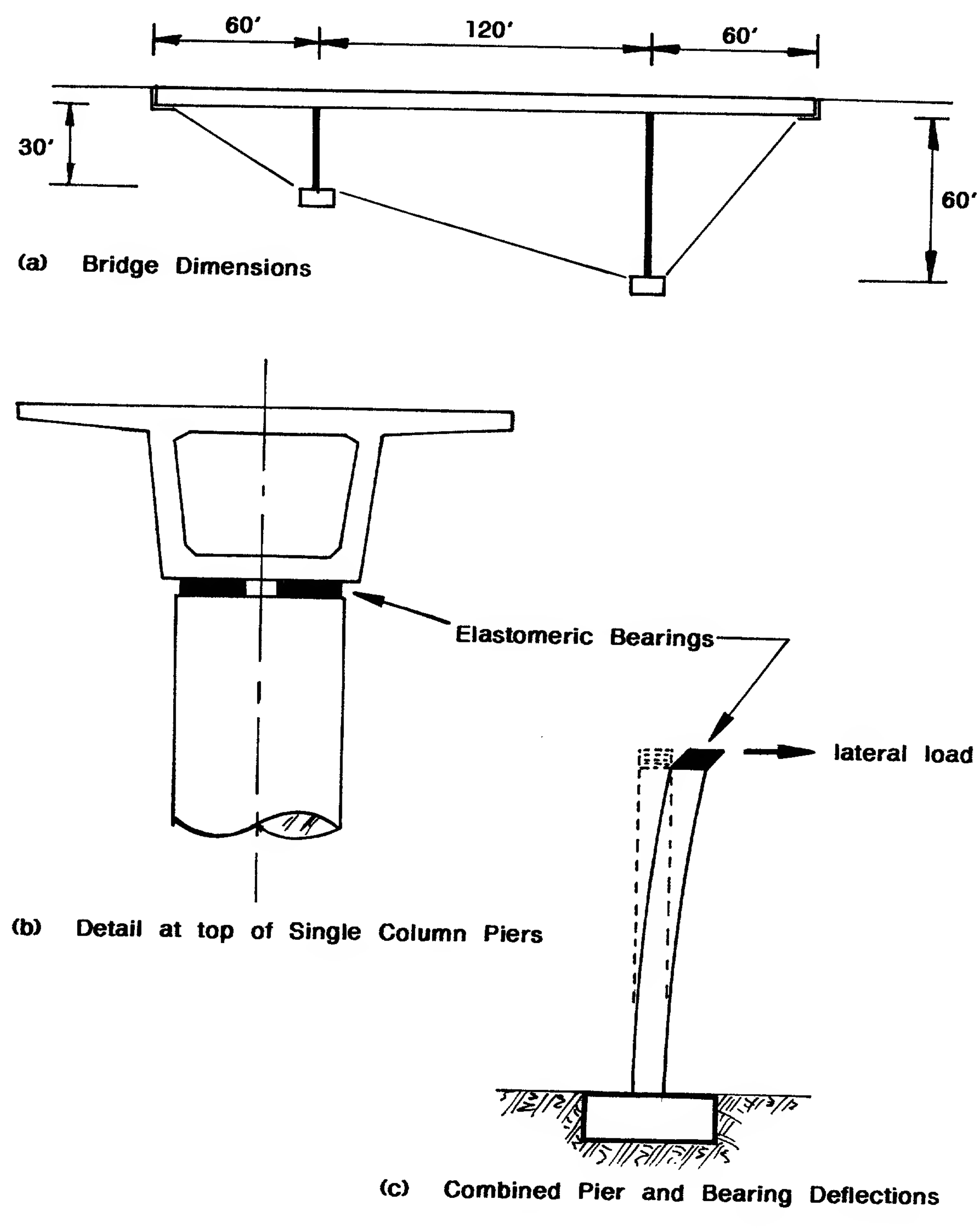


Figure 50: Effect of Elastomeric Bearings on Pier Lateral Stiffness

(d) Calculations

ASSUME: dead load per pier = 7K/ft x 120 = 840K
single column piers (6ft. dia.) I_c = $64ft^4$
modulus of elasticity of concrete E = 6000 ksi
shear modulus of elastomer G_r = 0.1 ksi
allowable compressive stress in bearings = 0.5 ksi
number of bearings on each pier = 2

THEN area of elastomer required for
given dead load = 840/0.5 = 1680 in^2
Since 2 bearings are to be provided on each pier,
each bearing should be 30 x 30 inch square
(A_r = 900 in^2) to carry the dead load.

NOW FROM FIGURE 47
stiffness of pier acting as a cantilever, $K_p = 3EI_c/h^3$
and stiffness of elastomeric bearing in shear, $K_b = G_rA_r/T_r$
therefore the shear stiffness of a pair of bearings
with 2-1/2, 5 and 7-1/2 inches of elastomer will be

T_r	K_b (for two bearings)
2-1/2	864 K/ft
5	432 K/ft
7-1/2	288 K/ft

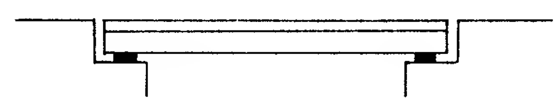
THEREFORE the combined stiffness of the single column pier
and 2 bearings will be given by:

$$K_T = \frac{K_b \cdot K_p}{K_b + K_p}$$

K_T for various combinations of columns and bearings are given below:

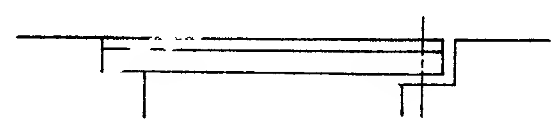
BEARINGS (all are 30 x 30 ins sq.)	COMBINED LATERAL STIFFNESS		RATIO
	Column Height = 30 (K/ft)	Column Height = 60 (K/ft)	$\frac{K_{30}}{K_{60}}$
none	6144	768	8.0
2-1/2 in. thick bearings on both columns	757	407	1.86
5 in. thick bearings on both columns	404	276	1.46
5 in. thick bearing on 30 ft col. 2-1/2 in. thick bearing on 60 ft col.	404	407	.99
7-1/2 in. thick bearing on 30 ft col. 5 in thick bearing on 60 ft col.	275	276	1.00
2-1/2 in. thick bearing on 30 ft col. no bearing on 60 ft. col.	757	768	.99

Figure 50: Effect of Elastomeric Bearings on Pier Lateral Stiffness (continued)



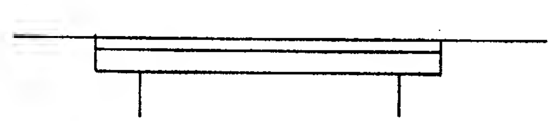
Description
both abutments seat-type with elastomeric bearings
road joints at both abutments

Comment
elastic restraint both directions at both abutments
load distribution in both directions controlled
by stiffness of elastomeric bearings



Description
monolithic LH abutment
seat-type RH abutment with sliding bearing
transverse shear keys RH abutment

Comment
longitudinal restraint LH abutment only
transverse restraint both abutments



Description
both abutments monolithic, no bearings
no road joints

Comment
restraint in both directions at both abutments

Figure 51a: Examples of Acceptable Structural Form for Single Span Bridges

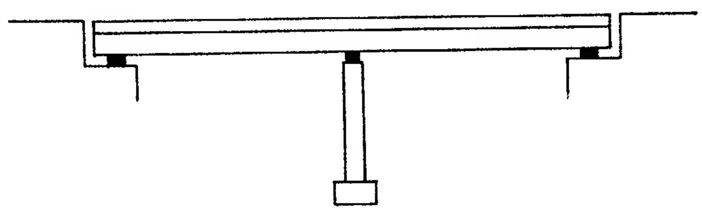
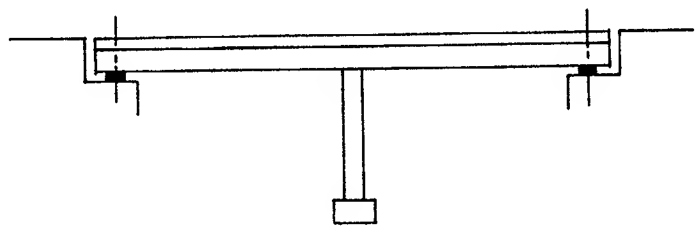
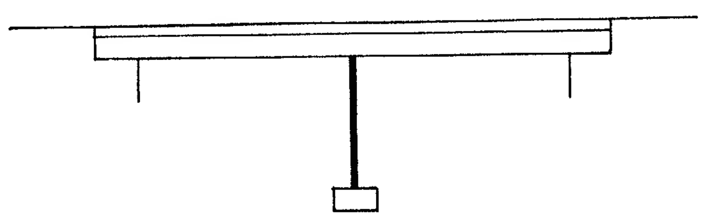
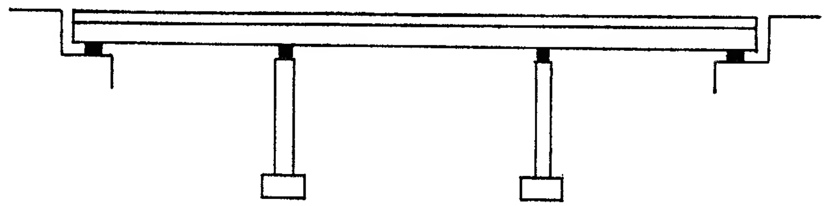
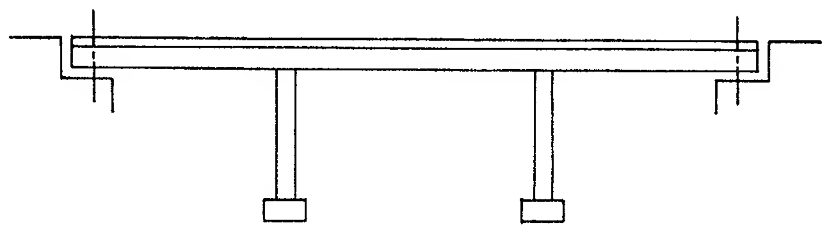
	**Description** both abutments seat-type with elastomeric bearings stiff columns both directions with elastomeric bearings road joints both abutments **Comment** elastic restraint in both directions at all supports possible plastic hinging base of columns load distribution in both directions controlled by stiffness of elastomeric bearings
	**Description** both abutments seat-type with elastomeric bearings stiff columns both directions, monolithic with deck transverse shear keys both abutments **Comment** elastic longitudinal restraint all supports transverse restraint at all supports expect plastic hinging base of columns
	**Description** both abutments monolithic column flexible longitudinally, monolithic with deck **Comment** longitudinal restraint at both abutments transverse restraint at all supports

Figure 51b: Examples of Acceptable Structural Form for 2-Span Bridges



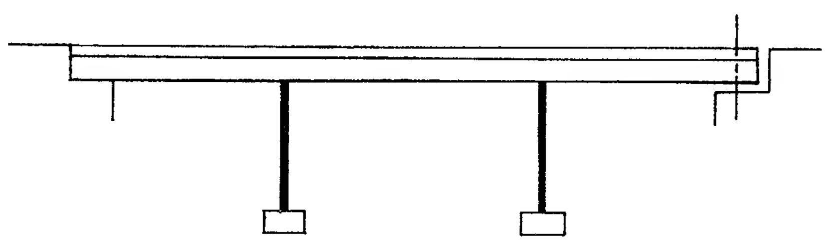
Description
both abutments seat-type with elastomeric bearings
stiff columns both directions with elastomeric bearings
road joints both abutments

Comment
elastic restraint in both directions at all supports
possible plastic hinging base of columns
load distribution in both directions controlled
by stiffness of elastomeric bearings



Description
both abutments seat-type with sliding bearings
and transverse shear keys
stiff columns both directions monolithic with deck

Comment
longitudinal restraint only in columns
transverse restraint at all supports
expect plastic hinging base of columns



Description
monolithic LH abutment
seat-type RH abutment with slider and
transverse shear keys
columns flexible longitudinally, monolithic with deck

Comment
longitudinal restraint LH abutment only
transverse restraint at all supports

Figure 51c: Examples of Acceptable Structural Form for 3-Span Bridges

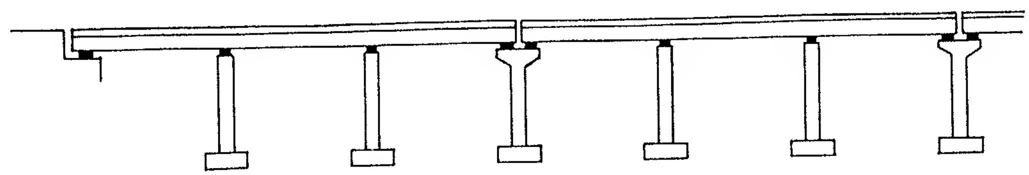
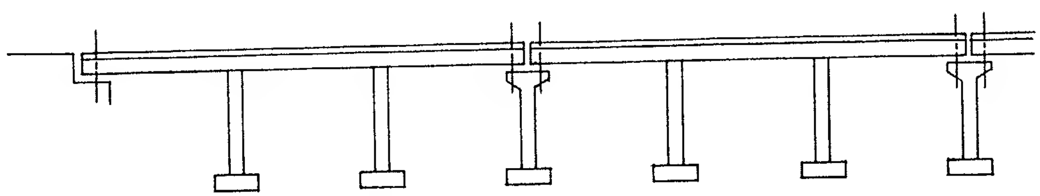
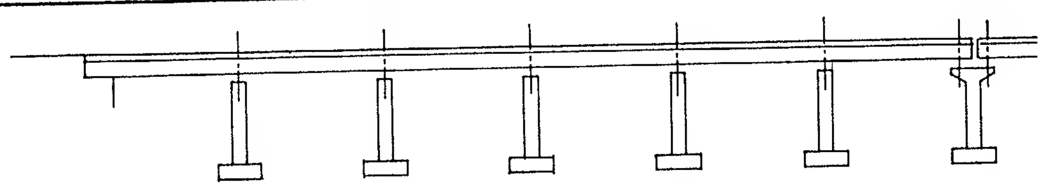
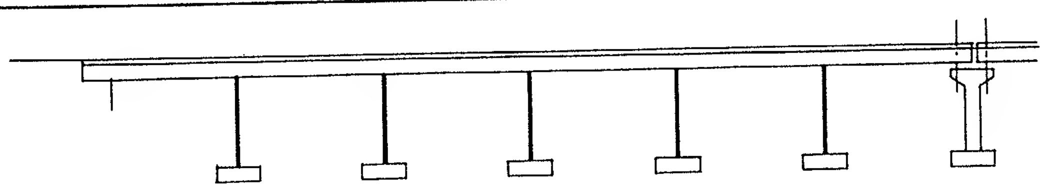
	**Description** abutments are seat-type with elastomeric bearings stiff columns both directions with elastomeric bearings expansion joints of bearings require double sets **Comment** elastic restraint in both directions at all supports possible plastic hinging at base of columns load distribution in both directions controlled by stiffness of elastomeric bearings
	**Description** abutments are seat-type with sliding bearings and transverse shear keys stiff columns both directions, monolithic with deck expansion joint requires double set of sliding bearings and shear keys **Comment** longitudinal restraint only in columns transverse restraint at all supports expect plastic hinging at base of columns
	**Description** abutments are monolithic stiff columns both directions with sliding bearings and transverse shear keys **Comment** longitudinal restraint at abutments only and very high forces are attracted into abutments transverse restraint at all supports
	**Description** abutments are monolithic columns flexible longitudinally, monolithic with deck expansion joint requires double set of sliding bearings and shear keys **Comment** longitudinal restraint at abutments only and very high forces are attracted into abutments transverse restraint at all supports

Figure 51d: Examples of Acceptable Structural Form for Multiple Span Bridges

forms is not intended to be all inclusive as other combinations and forms are possible which also give acceptable results.

5.1.6 Examples of Structural Form to be Avoided

To complement the previous section, figure 52 illustrates structural forms which are considered to be undesirable from a seismic point of view and should be avoided whenever possible.

The various examples are presented in 3 groups:

- longitudinal configurations to be avoided (figure 52a,b)
- transverse configurations to be avoided (figure 52c,d)
- pier configurations to be avoided (figure 52e,f).

Again, these examples are not all inclusive but are simply meant to be representative of poor structural form. It is also true that in many cases these undesirable structures can be upgraded to "acceptable" by relatively straightforward means.

5.2 UNUSUAL BRIDGES

This section reviews the seismic performance and design considerations for bridges of either unusual geometry or unusual form. Some notes on bridges in difficult sites complete this short review. This discussion is taken from reference 29 and has been modified where appropriate to suit U.S. conditions and practices.

5.2.1 Bridges of Unusual Geometry

Bridges of conventional structural type but unusual or extreme geometry require special care during design. The effect of extreme curvature, skew, height, length or width on seismic response should be considered and special precautions taken to avoid premature damage. This may take the form of performing more sophisticated analyses, taking special care with detailing and being more conservative when estimating forces and deformations. Further, the importance and/or exceptional cost of these bridges may justify a site-specific study to determine a design spectrum appropriate for the site and the bridge. This spectrum would then be used instead of the lateral seismic coefficient as given in the various codes and specifications.

A. Bridges with Severe Curvature

The seismic response of multispan bridges with large horizontal curvature and/or a large horizontal deflection angle is difficult to predict with accuracy, particularly when the superstructure is torsionally stiff, as will generally be the case, to improve live-load distribution. Full three dimensional modelling and analysis may be necessary to assure accuracy of force and deflection calculations. Multimode response will in general be important and results from single mode methods should be viewed with caution (see chapter 10).

Significant axial seismic forces may be induced in individual piers, even when each pier consists of a single column. Hinging may occur at the top of single-column piers cast monolithic with the superstructure.

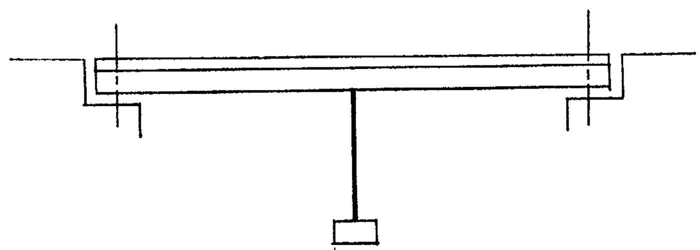
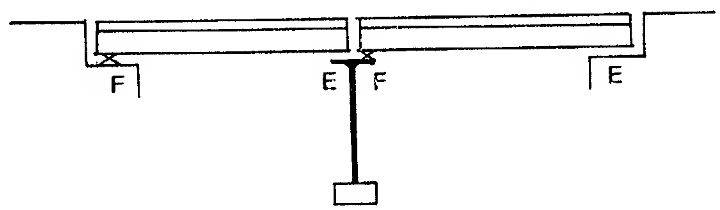
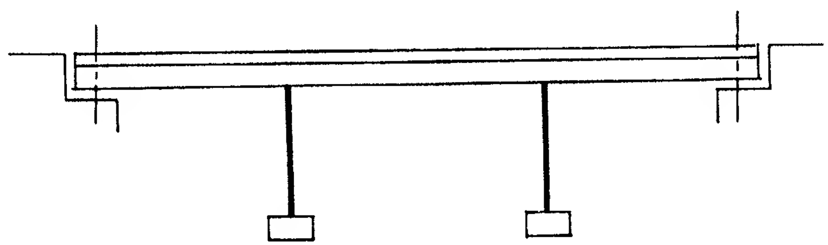
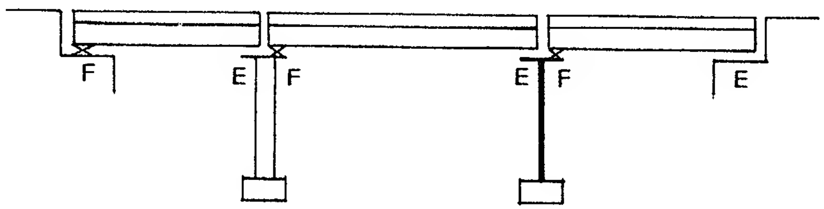
	**Description** both abutments seat-type with sliding bearings and transverse shear keys both columns are flexible longitudinally **Comment** superstructure will impact abutment walls **Remedy** stiffen columns and expect plastic hinging at base of each column
	**Description** both abutments seat-type column is flexible longitudinally pinned bearings located at F (no sliding) slide bearings located at E **Comment** no longitudinal restraint for RH span RH span will impact RH abutment LH span will fall from column support **Remedy** interchange F and E bearings RH span and provide generous seat widths/cable restrainers
	**Description** both abutments seat-type with sliding bearings and transverse shear keys column is flexible longitudinally, monolithic with deck **Comment** superstructure will impact abutment backwalls **Remedy** stiffen column and expect plastic hinging at base of column

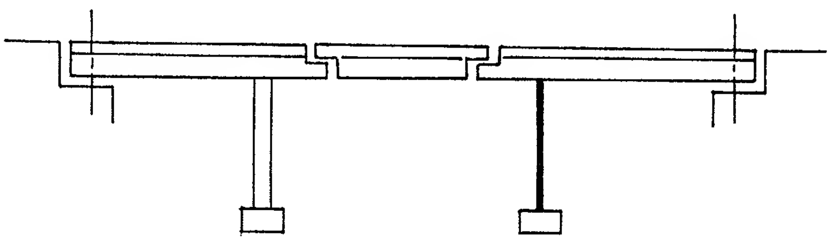
Figure 52a: Examples of Structural Form to be Avoided for Longitudinal Response of Bridges



Description
both abutments seat-type
LH column is stiff longitudinally
RH column is flexible longitudinally
pinned bearings are located at F
slider bearings are located at E

Comment
no longitudinal restraint for RH span
RH span will impact RH abutment
center span will fall from RH support

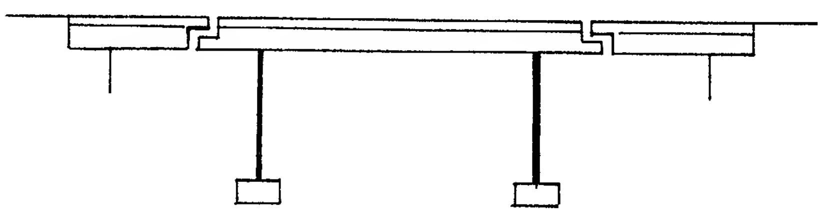
Remedy
interchange F and E bearings RH span and provide generous seat widths/cable restrainers



Description
both abutments seat-type with slider bearings and transverse shear keys
LH column is stiff longitudinally, monolithic with deck

Comment
no longitudinal restraint for RH span
RH span will impact RH abutment
center suspended span will fall from RH support

Remedy
tie superstructure together using cable restrainers or similar through both expansion joints



Description
both abutments monolithic
both columns flexible longitudinally, monolithic with deck

Comment
center section will impact end sections at each expansion joint

Remedy
tie superstructure together using cable restrainers or similar through one or both joints

Figure 52b: Examples of Structural Form to be Avoided for Longitudinal Response of Bridges (continued)

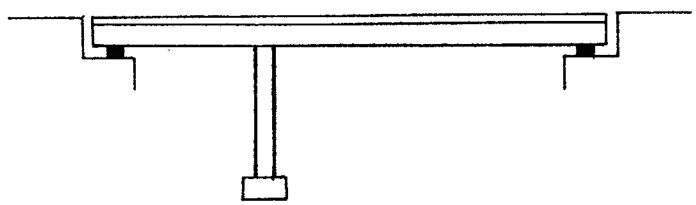
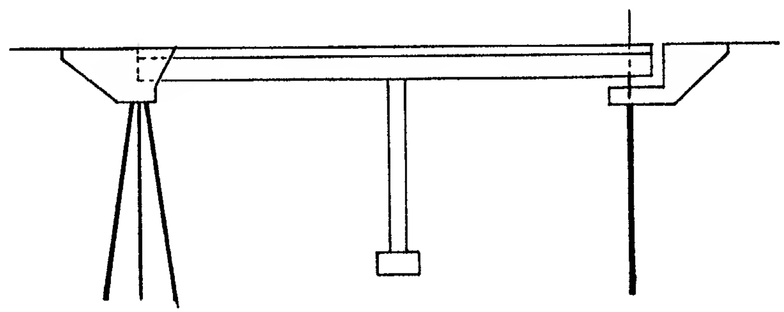
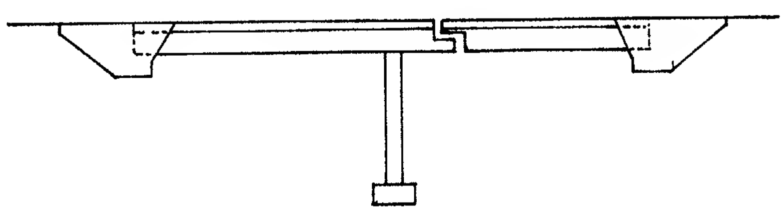
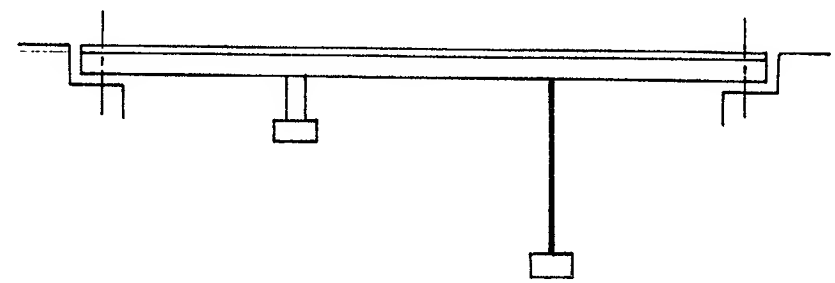
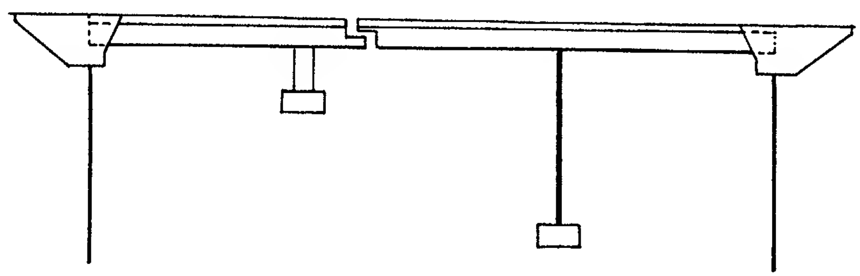
	**Description** both abutments seat-type with elastomeric bearing stiff column is off-center, monolithic with deck **Comment** lack of symmetry will cause twisting **Remedy** place transverse shear keys at both abutments
	**Description** LH abutment monolithic on batter piles RH abutment seat-type with sliding bearing and transverse shear keys, on vertical piles stiff column is monolithic with deck **Comment** stiffer piling system under LH abutment will cause twisting about LH abutment **Remedy** change multicolumn bent for a wall pier to reduce torsional rotations
	**Description** both abutments monolithic stiff column is monolithic with deck discontinuous superstructure with expansion joint In RH span **Comment** articulated deck will impose torsion on column and abutment structures **Remedy** change multicolumn bent for wall pier to reduce torsion; add restrainers across joint to provide some continuity

Figure 52c: Examples of Structural Forms to be Avoided for Transverse Response of Highway Bridges



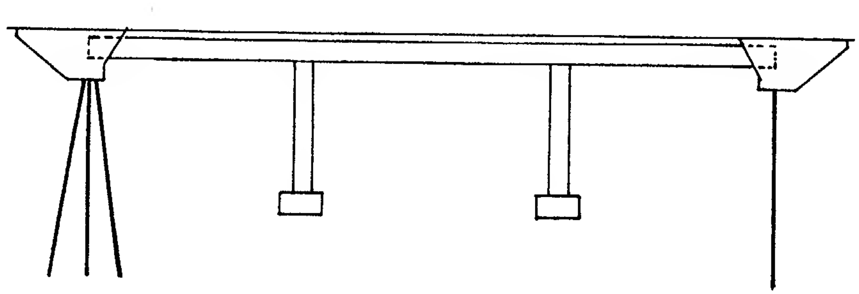
Description
both abutments seat-type with sliders and transverse shear keys
LH column stiff, RH column flexible longitudinally
Comment
although both columns stiff transversely, shorter column is stiffer and will attract higher loads
brittle shear failure in short column possible

Remedy
change RH column to wall pier to provide more uniform distribution of stiffness



Description
both abutments monolithic
LH column stiff, RH column flexible longitudinally
discontinuous superstructure with expansion joint in middle span
Comment
articulated deck will impose torsion on column

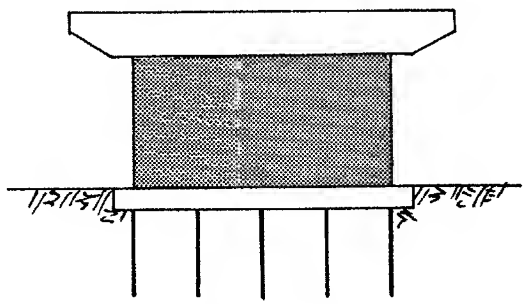
Remedy
change multicolumn bents for wall piers, especially RH column, to reduce torsion; add restrainers across joint to provide some continuity



Description
both abutments monolithic
LH abutment on batter piles,
RH abutment on vertical piles
both columns stiff and monolithic with deck
Comment
stiffer piling system under LH abutment will cause twisting about LH abutment

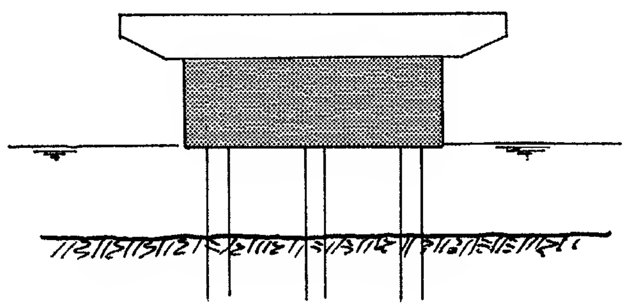
Remedy
change RH multicolumn bent for a wall pier to reduce torsional rotations

Figure 52d: Examples of Structural Forms to be Avoided for Transverse Response of Highway Bridges (continued)



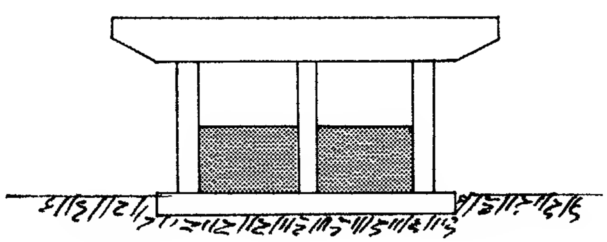
Description
wall pier founded on flexible vertical piles

Comment
high shear loads in wall impose high loads on piles
plastic hinges in piles will be underground and difficult to inspect and repair



Description
wall pier above water line in river crossing, piles extend above river bed to bottom of wall

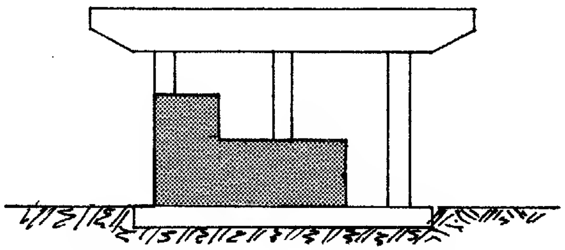
Comment
high loads in wall impose high shear forces and moments into unsupported sections of piles
plastic hinges will form underwater and be difficult to inspect and repair



Description
infill panels between columns of a multicolumn bent

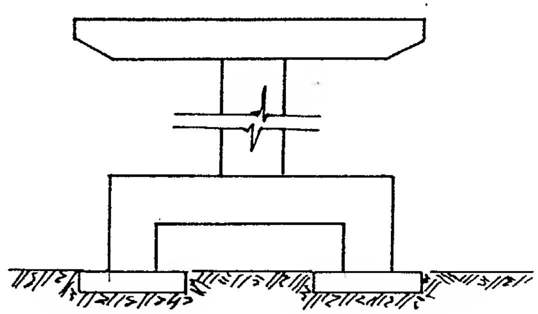
Comment
panels cause unintended change in stiffness and can force hinging into parts of the columns not detailed for ductile performance

Figure 52e: Examples of Structural Forms to be Avoided for Bridge Piers



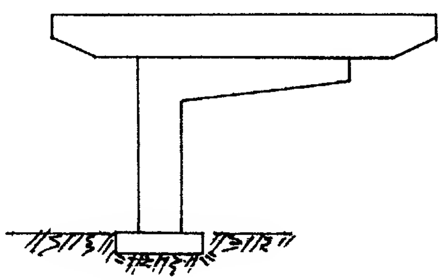
Description
each column of a multicolumn bent is supported on a wall of different height

Comment
the shorter column will attract most of the shear load and may fail in shear before flexural ductile action can develop in the bent



Description
single column is supported on two column bent

Comment
longitudinal loads will cause bending about weak axis and corresponding twisting moments in beam may cause nonductile failure of beam before flexural hinge can develop at base of single column



Description
eccentric single-column bent

Comment
high flexural demand at base of column (moments due to gravity and transverse seismic loads additive)
longitudinal response will impose torsional moments on column

Figure 52f: Examples of Structural Forms to be Avoided for Bridge Piers (continued)

The direction of relative movements or restraining forces at joints in the superstructure will be uncertain, and allowance for this should be made in detailing.

B. **Bridges with Severe Skew**

Skew bridges tend to rotate about a vertical axis during an earthquake. In both the San Fernando earthquake of 1971 and the Eureka earthquake of 1979, there was evidence of severe column damage due to this rotation and the consequential torsional shear forces. Abutments were also damaged as a result of increased longitudinal displacements and inadequate seating lengths. Figure 23 shows the collapse of two spans of a bridge during the Eureka earthquake due to this phenomenon.

Although there is some theoretical evidence to support this behavior the mechanisms are not clearly understood. Consequently, conservative assumptions should be made regarding necessary clearances, seating lengths, lateral restraint at abutments, and seismic shear forces for the intermediate piers. To survive these torsional moments and shears and the increased flexural ductility demand, conservative detailing in the columns should be adopted. For example, confining steel should extend over the full height of the columns, and not just be located in the plastic hinge zones.

C. **Bridges with Tall Piers**

Tall piers may be subjected to high inertial forces due to the response of the distributed mass of the pier itself. Ductility demands at intended plastic hinge locations may be substantially increased, and there may be a potential for plastic hinging to occur at a location close to mid-height, if the columns have substantial self weight. If preliminary analyses based on elastic response of distributed mass systems indicate the possibility of such behavior, the only realistic analysis will be a full dynamic inelastic time history analysis of the complete bridge system. Tall columns should be checked for additional moments due to $P-\Delta$ effects. Results of such an analysis may be highly dependent on stiffness values assumed for the columns. Because of a need to reduce inertial response, lighter framed bent systems might be preferable to heavier wall pier systems.

If the superstructure of a bridge in this category is restrained transversely at the abutments, the bending of the superstructure in the horizontal plane may be critical due to large lateral displacements of the piers. Similarly, for single column bents, torsion of the superstructure may be critical as a result of rotation of the column top when displaced laterally by a large amount.

D. **Bridges with Piers of Differing Heights**

When bridges span steep sided river valleys, substantial differences in adjacent column heights may be inevitable. Where possible, the stiffnesses of the substructures should be adjusted to result in as uniform a distribution of stiffness as possible.

Where such attempts to "regularize" the structural response are impractical, analyses must establish the realistic mass distribution to each substructure, and how this is influenced by sequential, rather than simultaneous yielding of the separate bents.

particularly in the longitudinal direction. The results of a carefully detailed elastic analysis can provide the basis for making this assessment but for bridges of unusual size and/or importance, an inelastic analysis may be necessary. For multispan bridges, transverse flexibility of the superstructure will generally become significant, particularly after initial yielding of some columns. Lateral forces transmitted back to abutments may be substantially different from those predicted by elastic analysis.

Before engaging in such analyses, it might be preferable to consider modifying the substructures to lessen the effect of the tall piers. This has been discussed earlier in section 5.1.4 and illustrated in figure 50. Possibilities include introducing pins at the base of the shorter columns, using batter piles under the taller piers, or placing elastomeric bearings at the top of one or more columns. On the other hand, it may be easier to make all substructures rigid and use energy dissipating devices to obtain ductility. Elastomeric bearings on all substructures would also be necessary here.

E. **Bridges with Long Continuous Spans**

Although the inherent flexibility of these structures should result in satisfactory performance, transverse superstructure deformations will become significant in bridges where the total length/width ratio is high. Mode shapes and forces induced in the support systems will be influenced by this flexibility, and superstructure plastic hinging may occur in extreme cases. Horizontal forces and displacements should be based on realistic estimates of transverse and longitudinal mode shapes and analysis should at least be on the basis of a multi-mode response spectrum approach.

Response of the superstructure to vertical acceleration should also be examined.

F. **Bridges with Long Discontinuous Spans**

The response of long bridge superstructures separated into two or more sections by expansion joints can be difficult to predict. Out-of-phase ground movements, as well as structural differences between the separate sections may cause substantial relative movements across these joints.

For longitudinal response it will generally be conservative to assess seismic forces and ductility demands on columns by considering each section independently. Maximum feasible relative displacements at the expansion joints could be obtained by considering peak displacements of the adjacent sections to be out of phase. However, a more realistic estimate of forces and relative longitudinal displacements may be obtained by modelling the expansion joint as a spring system to represent the combined shear stiffness of the bearings and the axial stiffness of seismic linkage bolts or cable restrainers. Out-of-phase ground movements at supports may add to relative displacements and should be considered.

A serious disadvantage of the spring representation for joint behavior is that it does not model impact as the joint closes. However, the overall response of the structure is expected to be adequately modelled using springs, but the detail design of the restrainers and bearings should use forces calculated by alternate procedures, such as those outlined in section 9.5.1.

The expansion joint will act as a hinge for transverse seismic response. The resulting loss of stiffness to the superstructure may cause an increase in displacements, and hence increased ductility demand to the substructures on either side of the joint. Realistic modelling of the articulation is necessary to ensure adequate prediction of transverse response. Such a model should include the restraining forces at the joint provided by the bearings and linkage systems, and the possibility of impact associated with total joint closure.

G. **Bridges with Substructures in Deep Water**

The response of bridge substructures in deep water is affected by hydrodynamic mass of a volume of water being forced to move with the pier. A reasonable estimate of the hydrodynamic added-mass is the mass of a circular cylinder of water of diameter equal to the column width perpendicular to the direction of motion, and length equal to immersed depth. This mass should be added to the substructure mass when considering the seismic response.

5.2.2 Bridges of Unusual Type

Typical highway bridges have either steel or concrete girder superstructures and rely on columns and foundation structures to provide resistance and ductile response to earthquakes. Because of their wide use and occurrence, these girder bridges have been the focus of most of the discussion so far. But at least three other bridge types also deserve consideration, particularly in view of their importance for longer span bridges. These are the trusses, bridges with cable-supported decks and arch bridges.

A. **Trussed Superstructures**

The steel truss was frequently used for medium-span bridges until it was overtaken by the more economic and aesthetically pleasing box girder. From a seismic point of view, there is almost no difference in performance between a truss and a girder superstructure. Since the substructure dictates seismic performance, and these are usually the same for both bridge types, similar behavior can be expected. The inherent in-plane stiffness of a 3-dimensional truss will assure adequate lateral load distribution to the piers and abutments and no special considerations are therefore necessary for this class of bridge.

B. **Cable-Supported Superstructures**

Suspension and cable-stayed bridges have structurally complex superstructures. Because of the typically long spans involved, superstructures are relatively flexible both vertically and transversely. In assessing transverse and longitudinal seismic forces induced in superstructure elements or transmitted to supporting piers, realistic estimates of mode shapes, including consideration of flexibility of the superstructure, must be adopted. It is probable that as a result of long fundamental periods, response will be dominated by higher mode effects. As the superstructure will probably be required to remain elastic during the design earthquake, analysis may consist of an elastic modal analysis approach. However, at high amplitudes of vibration, nonlinear behavior due to large deflections in both suspension and cable-stayed bridges will tend to invalidate most modal techniques. Low damping can be expected from elastic response of continuous steel superstructures in the

longitudinal and transverse directions. For analysis it is recommended that a value of 2 percent of critical damping be assumed.

In the vertical direction, cable-stayed bridges may have high damping due to unequal cable lengths, and the non-linear load-displacement characteristics of the bridge. If the displacements are of small amplitude and response essentially linear, then this large damping may not be apparent. Vertical response of suspension bridges in the higher modes may have low damping, and should be considered.

There is little information available on the ductility of such large elements as are commonly used for piers of cable-supported bridges. Consequently such piers should be designed for low ductility levels, or if possible, to remain elastic under the design earthquake. Because of large distances between major piers, large out-of-phase displacements may occur. Although the superstructure flexibility is likely to be such that these displacements can be easily accommodated, checking is necessary. Analyses should be based on a relative longitudinal or transverse displacement of adjacent major piers equal to twice the maximum response displacement.

C. **Arch Bridges**

It is difficult to detail arch bridges for ductility, and, where possible, they should be designed to respond elastically to the design earthquake. However, this may be too restrictive for some arch bridges and elastic response may not be economic. This requirement might therefore be relaxed if the structure is of steel, and perhaps also for those concrete arches which can be designed to exhibit some ductile response. A detailed structural analysis may be necessary to define longitudinal and transverse mode shapes.

Special consideration must be given to relative longitudinal displacement of the arch springing due to out-of-phase ground motion and seismic response displacements of the typically steep embankments. Detailed geotechnical investigations should be carried out to establish the integrity and stability of the embankments under seismic conditions.

5.2.3 Bridges in Difficult Sites

Whenever possible, bridges should not be sited in locations where adverse ground conditions significantly increase seismic risk. Such locations include sites crossing or immediately adjacent to an active fault, steep slopes with potential instability under earthquake conditions, and sands with a potential for liquefaction.

A. **Sites Across or Near Active Faults**

Bridges crossing or immediately adjacent to active faults may be subjected to large relative displacements of adjacent piers or supports as a result of surface faulting. Although the probability of such occurrence at a given location during the design life of the bridge will be very low, the possibility should be considered in assessing a suitable structural type. A conservative design, particularly in terms of displacement capabilities should be adopted. Design of substructures should aim at providing the maximum capacity possible, by use of extra confinement in the plastic hinge zones. It may be advisable to provide an inner confined core capable

of supporting the structural dead weight on the assumption that the outer flexural confinement will have failed under an extreme event. This has the added advantage that under moderate, though not catastrophic inelastic displacements, the piers may be repaired by cutting out and replacing the buckled outer layer of steel.

Whenever possible, single span, low level crossings of active faults are preferred. However, if multispan crossings are necessary, the relative merits of continuous construction compared to simple spans should be carefully evaluated. Although simple spans have the advantage of additional flexibility, difficulty will be experienced in ensuring that the spans do not drop from supports. To minimize this risk, very generous support lengths should be provided. The additional redundancy of continuous superstructures which are monolithic with their supporting substructures will tend to reduce the probability of total collapse. There is, however, a practical limit to the amount of relative displacement across a fault that can be accommodated in a monolithic structure. One alternative is to support a continuous superstructure on elastomeric bearings over each pier and at each abutment. These bearings can be designed to accommodate relatively large displacements and still provide an elastic restoring force to the superstructure. Additional restrainers may also be provided in parallel with the bearings if gross movements are expected. Note that accelerographs of recent earthquakes indicate that vertical ground accelerations close to a fault can substantially exceed 1.0g. In these situations, monolithic construction is to be preferred but if elastomeric bearings are used, vertical restrainers should be provided to limit the effects of uplift.

It should be recognized that the purpose of design for such an extreme event will be to avoid, or at least minimize, loss of life by reducing the probability of total collapse. After such an earthquake it is probable that the bridge will have to be demolished and replaced.

B. **Slopes with Instability Potential**

Many bridges are inevitably sited across steep-sided valleys. Detailed geotechnical investigations should be made to assess potential for slope instability under seismic attack. For major structures these investigations should include geological and geomorphic studies including expert study of aerial photographs, for evidence of bank movement under recent earthquakes, as well as material testing and extensive bore-hole and trenching investigations to check for unstable layers and vertical fissures. Particular attention should be paid to drainage to prevent infiltration of surface water and increased porewater pressures in potential failure regions. Special studies should be made to investigate the practicality of improving factors of safety against slope failure using such means as unloading the banks by removal of overburden. It may be advisable to site each abutment well back from the top of the slope, and tie back any intermediate pile caps located on the bank using rock anchors or other techniques.

C. **Liquefiable Foundations**

There does not appear to be any viable method to design a bridge to remain serviceable if liquefaction occurs under one or more of the substructures. As liquefaction can occur at considerable distances from the epicenter, the factor of safety against liquefaction must therefore be high. In rare cases when the factor of safety is felt to be only marginally acceptable, the design should aim to provide

maximum feasible ductility capacity to avoid total catastrophic collapse. Much of the above discussion concerning design for sites near active faults is applicable here also. But in contrast to the near-fault hazard, there are several techniques for reducing the probability of occurrence of liquefaction. These range from the use of stone columns to the removal of liquefiable material from the site. Liquefaction is discussed in greater detail in section 9.7 and references 4 and 6.

5.3 STRUCTURAL DETAILS

As illustrated in figure 46, there are four basic components to a typical bridge system. These are:

- Superstructure.
- Bearings/Joints/Shear Keys.
- Substructures.
- Foundation Structures.

It is the purpose of this section to review some typical configurations for each of these components and to highlight details which are considered important from a seismic point of view.

5.3.1 Superstructures

The most commonly used bridge superstructures are:

- Steel stringer bridges with concrete deck slabs.
- Precast prestressed concrete girder and slab bridges.
- Cast-in-place post-tensioned or reinforced concrete T-girder and box girder bridges.

Each type can be used for simple and continuous spans and can be made integral with abutments and supporting structures if desired. Today there is clear preference for reducing the number of joints in a superstructure, primarily for maintenance reasons, but also because of the improved seismic performance that accrues from continuity.

Also of importance is the rigidity of the superstructure to in-plane loads. Generally this is improved if there are end and intermediate diaphragms to restrain section distortion within the deck and to preserve composite action between the webs and flanges. Box girders are particularly attractive because of their high in-plane rigidity and their consequential ability to distribute loads back to the abutments. In very long span structures this property is less important since the superstructure will be flexible no matter what the section, but as a general rule, a superstructure with a closed or partially closed cross-section will behave better than one that is open and prone to distortion.

Sometimes, for reasons of economy in construction, precast simply supported spans are preferable even for multi-span structures. In these cases, it is possible to provide a measure of continuity for in-plane loads by making the deck slab continuous. To prevent imposing unintentional continuity for live load, the slab reinforcement is detailed to permit rotation about a horizontal axis near each support while still transfering in-plane forces from span to span.

5.3.2 Bearings, Joints, Shear Keys, Restrainers

Bearings are provided in continuous and simple span structures to permit relative movements to occur between the superstructure and the various substructures due to temperature, as well as accommodate shortening due to prestress (where provided) and creep and shrinkage (if a concrete structure).

Of particular importance, from the seismic point of view, is that the seating widths be generously proportioned so as to avoid spans falling from their supports. Guidelines for these support lengths are discussed in sections 7.4, 7.5 and 7.6.

Shear keys, bearings with side stops, and other restraining devices (such as cable systems) should be designed to transmit realistic earthquake forces elastically. In other words, these items of hardware, which are critical to the integrity of the structure, must have sufficient strength to remain elastic during the design earthquake for the site. Particular attention should be given to shear keys because failure in shear is usually brittle and catastrophic. These items must therefore be overdesigned and design forces which are, say, 20 percent higher than the elastic forces should be used. Postelastic behavior should only be considered for these elements during extreme earthquake events.

5.3.3 Substructures

Single columns, wall piers and multicolumn bents are the most common form of substructures. It is economically feasible to design columns to yield and dissipate significant amounts of energy and to perform in a ductile manner.

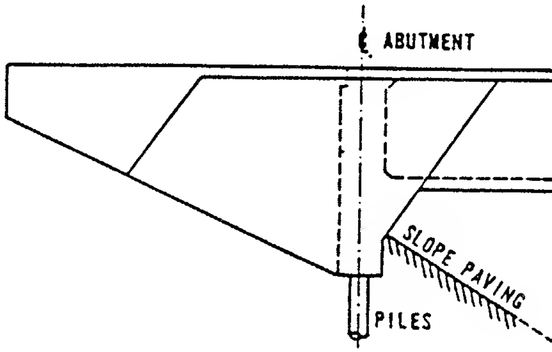
Wall piers are very much stiffer in the transverse direction than bent piers and generally force ductile action into the foundation structures. If these are piles, plastic hinging may occur at depth which could be difficult to inspect and repair. If piles are not used, rocking may occur under a wall pier which may then have undesirable consequences for the superstructure--uplift and torsional deformations will be imposed simultaneously.

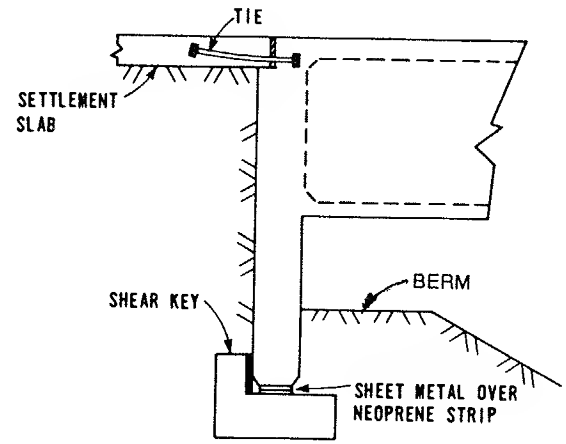
5.3.4 Foundation Structures

Footings and piles are the most common form of bridge foundation structure. Spread footings are economical and used wherever adequate soil conditions occur near the surface. Piles are generally used in groups and may be vertical or batter to transfer superstructure forces to load bearing soils at depth. However, drilled shafts are becoming increasingly popular because they are generally cheaper than multiplle foundations and can be used in congested locations with minimum disruption to existing services.

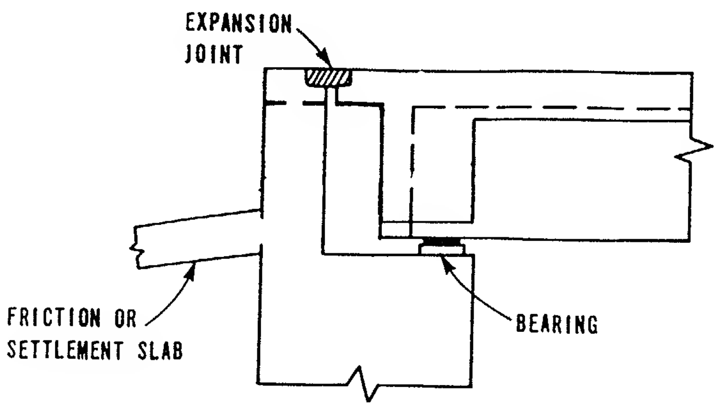
Two types of abutments are commonly used and these are illustrated in figure 53. These are the monolithic, or integral abutment, and the seat-type abutment. The essential difference is that one permits relative movement to occur between the superstructure and end support and the other does not.

Within the class of **monolithic abutments** there are several variants. Two of these are shown in figure 53a. Both use an end diaphragm to transfer forces into the abutment, but in one case the diaphragm is supported directly on vertical piles and in the other it rests on a strip footing.





(a) Monolithic abutments also called Integral or End Diaphragm Abutments



(b) Seat-Type Abutment

Figure 53: Typical Bridge Abutments

In the former situation, the vertical piles are usually sufficiently flexible to permit shortening of the superstructure due to prestress, shrinkage, creep and thermal effects. Reinforced concrete bridges up to 400 ft in total length have been successfully supported on piled abutments of this kind without evidence of distress. When piles are not necessary, a neoprene pad or similar material is provided to permit the relative horizontal movements to occur between the diaphragm and footing. This movement is virtually unrestrained in the direction away from the backfill but is limited to about 1/2 inch in the opposite direction, i.e. towards the backfill. Here a shear key is provided to assist in distributing the longitudinal seismic forces into the soil and to help mobilize the energy absorption capacity of the backfill. The small gap between shear key and end diaphragm permits thermal expansion to occur in the superstructure. To prevent soil falling into this gap, it is usually filled with an expansion joint filler compound. Shear keys are also provided at the ends of the footing to restrain transverse movement.

As noted above, monolithic abutments will mobilize the backfill under both longitudinal and transverse loads and can therefore dissipate or absorb significant amounts of energy during an earthquake. If it is intended to attract large forces into the abutment, this abutment type is to be preferred. However, damage to the end diaphragm, wing walls and piles may be severe because of the large forces to be resisted.

On the other hand, if elastic forces are used to proportion these components, overall collapse should be prevented. As an added precaution, the berm width should be generously proportioned so as to prevent total loss of support should the end diaphragm fail catastrophically.

The major attraction of the monolithic abutment is that it is economical to construct and maintain because of the absence of road joints and bearings. However, a serious problem can arise with these abutments. Whereas superstructure shortening can be accommodated in the abutment itself, the backfill and road surface may not follow the movements of the end diaphragm. The gap that forms behind the diaphragm permits the intrusion of water and can lead to the erosion of the slope below the abutment and severe washouts may be experienced. To minimize the risk of this occurring, limitations are usually placed on the total length of the structure that can be supported on monolithic abutments. These restrictions are more severe for cast-in-place prestressed concrete superstructures to allow for additional movements due to plastic shortening. Table 3 presents a set of maximum bridge lengths (as recommended by Caltrans), which should not be exceeded if monolithic abutments are to be used. Note that these are a function of temperature range and superstructure type and are intended to limit the maximum movement from all sources at one abutment to less than or equal to 3/4 inch.

Table 3: Recommended Maximum Bridge Lengths (feet) for Monolithic Abutments
(from Reference 30)

Temp. (°F) Range	SUPERSTRUCTURE TYPE Steel	Reinforced Concrete	Precast Concrete	CIP/Prestressed Concrete
80	240	260	240	150
100	200	210	200	130
120	160	180	170	120

It is also worth noting that Caltrans will not use monolithic abutments on a bridge that carries storm water by open channel flow [reference 31]. Bridges which are on a slope may be used by local authorities to channel uphill runoff over depressed freeways or similar obstacles. To avoid the flow of a large volume of water behind the end diaphragm, seat-type abutments are used with a flexible road joint seal at the abutment to ensure water tightness of the road and bridge surface.

In addition to providing a watertight seal, **seat-type abutments** also permit better control over the seismic forces imposed at the abutment and in general, they sustain less damage during an earthquake than a monolithic abutment. The joint can be adjusted in size from a few inches for thermal movement to a few feet for seismic effects. However, the cost of providing a road joint to accommodate seismic deflections of the order of 2-3 feet will be prohibitive and a compromise is usually adopted. A smaller gap (and cheaper road joint) is provided which is sufficient to accommodate thermal movements and small-to-moderate earthquakes. During a major earthquake (the design event) impact against the abutment wall is expected and some damage tolerated. There are various schemes to mitigate the extent of this damage which use a break-away section near the top of the wall. After failure, the superstructure has the necessary freedom to move. Temporary repairs can be completed quickly and permanent reconstruction of the damaged section is inexpensive [reference 32].

Regardless of the detail used in the joint itself, seat-type abutments must be provided with generous seat lengths. Recommendations for these dimensions are given in chapter 7. Seat abutments also reduce the redundancy in a bridge by the introduction of an extra joint. The associated increase in flexibility results in higher deflections but these can be accommodated in elastomeric or sliding bearings without difficulty. If the deflections become excessive, the generous seat lengths noted above should be adequate protection against loss of support.

CHAPTER 6 DESIGN LOADS

The detailed seismic design of a bridge is performed once a basic design concept (chapter 5) has been developed. An outline of the design process is given in figure 54 and involves as a first step an analysis of the structure using the appropriate design loads.

The magnitude of the design loads, determination of the component design forces and the associated detailed design requirements presented herein and in chapter 7 are a function of the design philosophy (chapter 4). Primary emphasis is given to the design philosophy and design requirements of the AASHTO Guide Specifications [reference 4].

In the development of the design loads, it must be emphasized again that the specification of earthquake ground shaking cannot be achieved solely by following a set of scientific principles, for the following reasons. First, the causes of earthquakes are still not well understood and experts do not fully agree as to how the available knowledge should be interpreted to specify ground motions for use in design. Second, to achieve workable bridge design provisions it is important to simplify the complex matter of earthquake occurrence and ground motions. Finally, any specification of a design ground shaking involves balancing the risk of that motion occurring against the cost to society of requiring that structures be designed to withstand that motion. Hence judgment, engineering experience and political wisdom are as necessary as scientific knowledge. In addition, it must be remembered that design ground shaking alone does not determine how a bridge will perform during a future earthquake. There must be a balance between the specified shaking and the rules used to assess structural resistance to that shaking.

In the AASHTO Guide Specifications, the design loads are expressed as a design coefficient or design response spectra (section 2.5 and 3.4) which represent the expected realistic force levels for the site. These force levels are derived such that they have a 10 to 20 percent chance of being exceeded every 50 years and are a function of the acceleration coefficient and the site soil conditions. The factors on which these design loads are based are discussed in section 6.1, 6.2 and 6.3 of this chapter. The component design forces are obtained by dividing the elastic forces, obtained from the analysis, by a Response Modification Factor, the basis of which is discussed in section 6.4. The detailed design requirements of the AASHTO Guide Specifications are a function of a bridge's Seismic Performance Category. These categories are discussed in section 6.5.

6.1 ZONING MAPS AND ACCELERATION COEFFICIENT

The first step in the determination of the design loads is the use of seismic zoning or regionalization maps to determine the zone in which the bridge site is located.

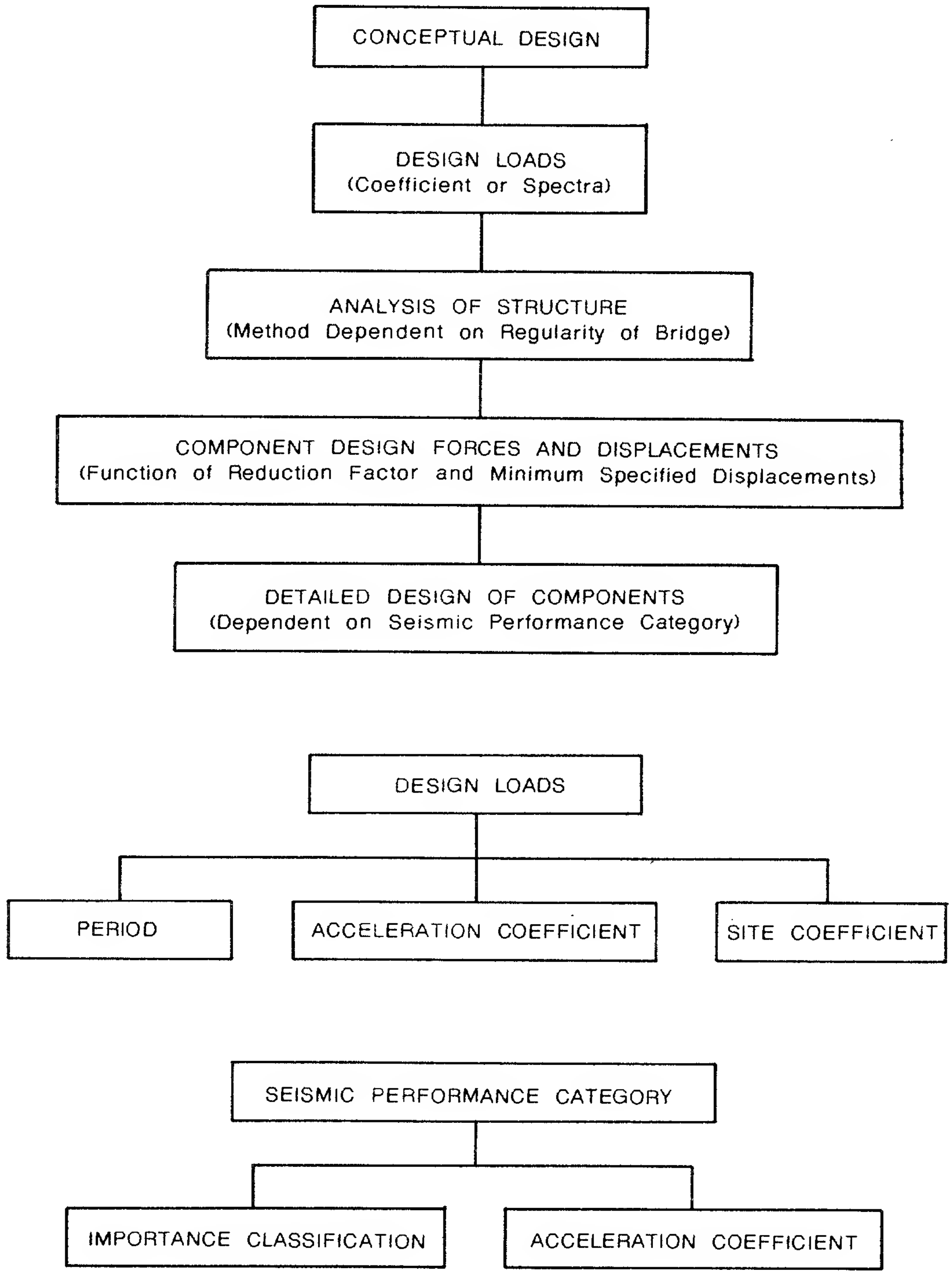


Figure 54: Design Process

This defines a level of seismic risk and it is useful to understand the basis on which these maps were developed. The following discussion is the basis for the Seismic Zoning or Regionalization Maps shown in figure 55. These maps are based upon two major policy decisions.

The first is that the probability of exceeding the design ground shaking should, as a goal, be assumed to be equal in all parts of the United States. The desirability of this goal is accepted within the profession. However, there is some disagreement as to the accuracy of estimates of probability of ground motion as determined from current knowledge and procedures. Use of a contour map based on uniform probability of occurrence is a departure from that implied in the zone maps of the AASHTO Standard Specifications [reference 1] shown in figure 56. These are based on estimates of maximum ground shaking experienced during the recorded historical period without any consideration of how frequently such motion might occur. It is also recognized that the real concern is with the probability of structural failures and resultant casualties and that the geographical distribution of that probability is not necessarily the same as the distribution of the probability of exceeding some ground motion. Thus, the goal as stated is the most workable one for the present but not necessarily the ideal one for the future.

The second policy decision is that the regionalization maps should not attempt to microzone. In particular, there is no attempt to locate actual faults on the regionalization maps, and variations of ground shaking over short distances--about 10 miles or less--are not considered. Any such microzoning must be done by qualified professionals who are familiar with localized conditions. Many local jurisdictions may find it expedient to undertake microzoning.

6.1.1 Design Earthquake Ground Motion

The determination of appropriate seismic design loads, although complex in reality, has been significantly simplified for code application. To state the concept rather than provide a precise definition, the design ground motion for a location is the ground motion that the engineer should consider when designing a structure to provide a specified degree of protection for life safety and to prevent collapse.

At present, the best workable tool for describing design ground shaking is a smoothed elastic response spectrum for single degree-of-freedom systems (sections 2.5 and 3.4). Such a spectrum provides a quantitative description of both the intensity and frequency content of ground motion. Smoothed elastic response spectra for 5 percent damping are used as a basic tool for the representation of local ground conditions.

6.1.2 Risks Associated with the Contour Map

The probability that the recommended Acceleration Coefficient and associated response spectra at a given location will not be exceeded during a 50-year period is estimated to be about 90 percent. At present, this probability cannot be estimated precisely. Moreover, since the maps were adjusted and smoothed by the project engineering panel of the Applied Technology Council, after consultation with seismologists, the risk may not be the same at all locations. However, it is believed that the probability of the design response spectra not being exceeded is in the range of 80 percent to 90 percent. The use of a 50-year interval to characterize the probability is a rather arbitrary convenience, and does not imply that all bridges are thought to have a useful life of 50 years.

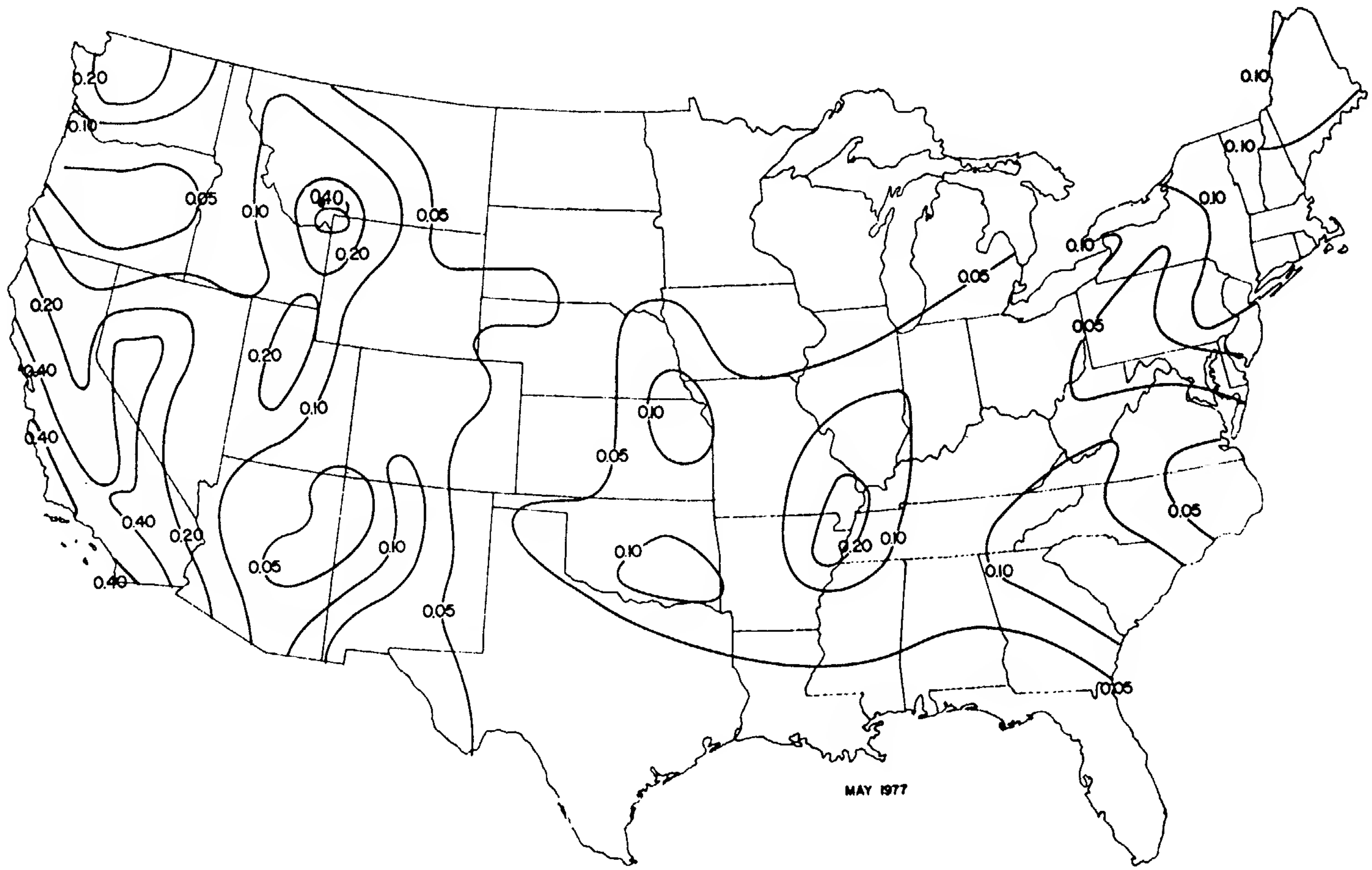


Figure 55: Acceleration Coefficient – Continental United States (from Reference 4)

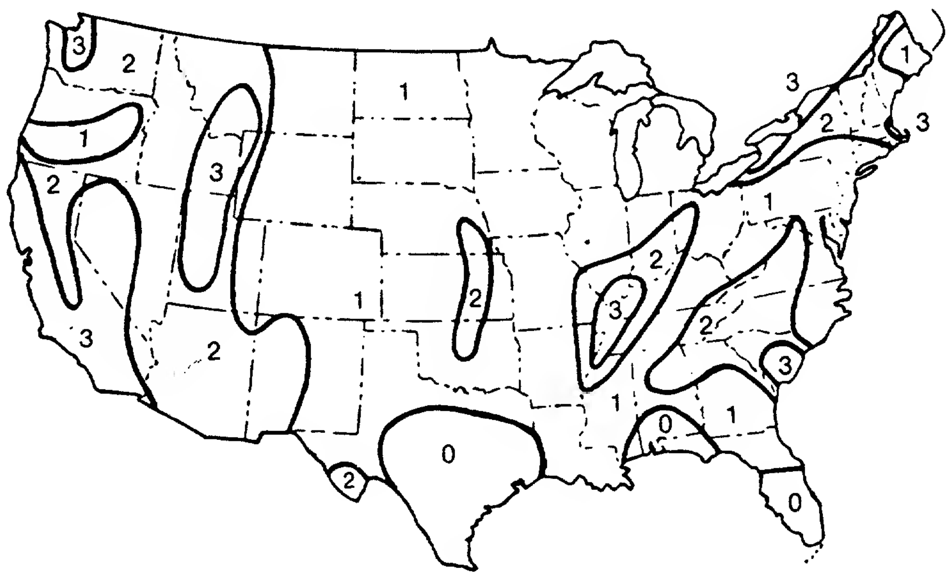


Figure 56: Seismic Risk Map of the United States (from Reference 1)

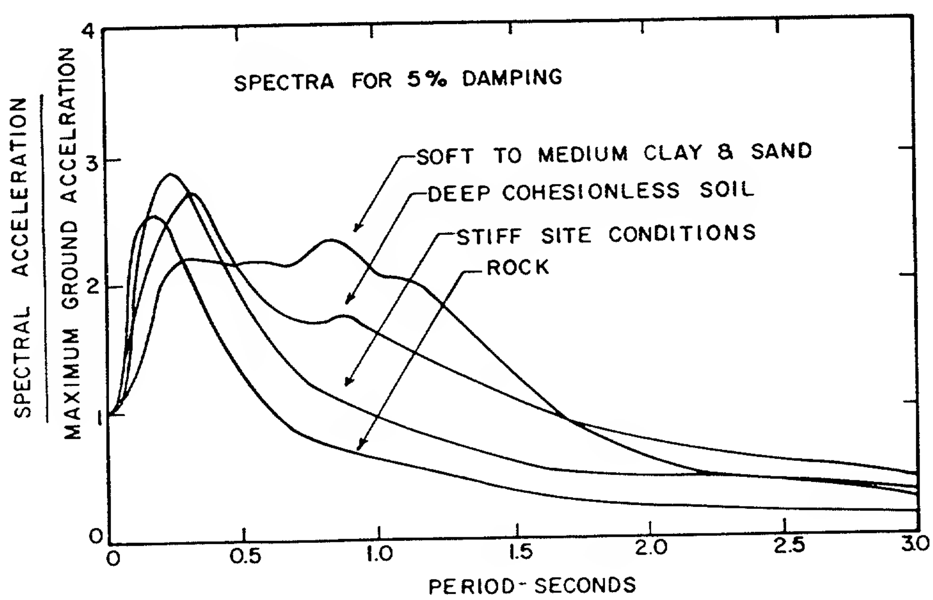


Figure 57: Average Acceleration Spectra for Different Site Conditions (from Reference 4)

6.2 INFLUENCE OF SOIL CONDITIONS ON GROUND MOTION

At the present time there is a widespread agreement that the characteristics of ground shaking and the corresponding response spectra are influenced by:

- The characteristics of the soil deposits underlying the proposed site (section 2.5).
- The magnitude of the earthquake producing the ground motions (section 2.4).
- The source mechanism of the earthquake producing the ground motions (section 2.2 and 2.3).
- The distance of the earthquake from the proposed site and the geology of the travel path (section 2.4).

While it is conceptually desirable to include specific consideration of all four of the factors listed above it is usually not possible to do so in a code environment because of the complexity of the problem. Sufficient information is available to characterize, in a general way, the effects of specific soil conditions on effective peak acceleration and spectral shapes. The effects of the other factors are so poorly understood at this time that they are often not considered in spectral studies.

The fact that the effects of local soil conditions on ground motion characteristics should be considered in structural design has long been recognized in many countries of the world. Most countries considering these effects have developed different design criteria for several different soil conditions. Typically these criteria use up to four different soil conditions.

In the AASHTO Guide Specifications, spectral shapes representative of four different soil conditions were selected from a study performed by Seed et al [reference 16]. The mean spectral shapes were based on 104 recorded ground motion records, primarily from earthquakes close to the seismic source zone in past earthquakes in the Western United States. The statistical mean of these shapes for the four different soil conditions are shown in figure 57. It was considered appropriate to simplify the curves to a family of three by combining the spectra for rock and stiff soil conditions; the normalized spectral curves are shown in figure 58. The curves in this figure thus apply to the following three soil conditions.

Soil Profile Type I is a profile with **either**:

1. Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 ft/sec (760 m/sec), or by other appropriate means of classification); **or**

2. Stiff soil conditions where the soil depth is less than 200 ft. (60 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

Soil Profile Type II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 200 ft. (60 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

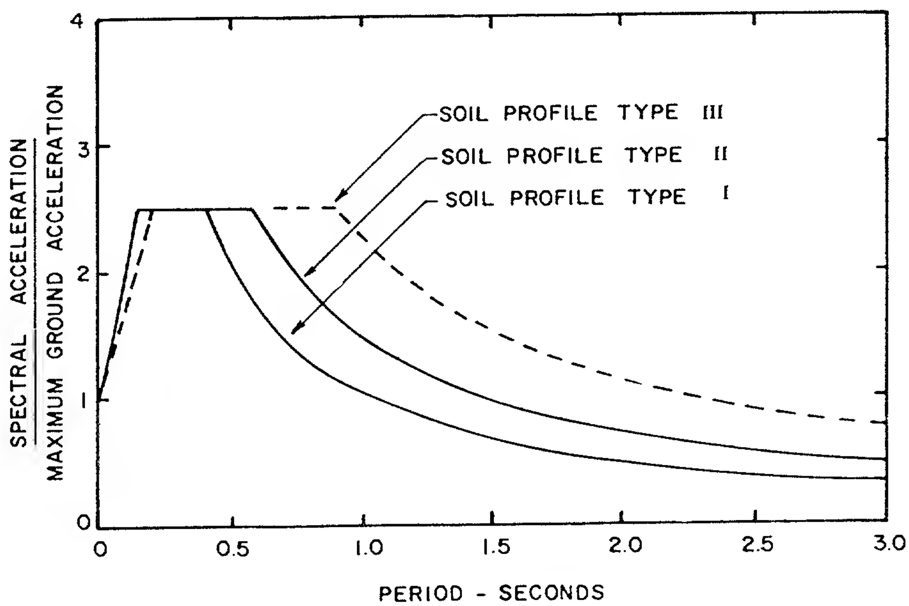


Figure 58: Normalized Response Spectra (from Reference 4)

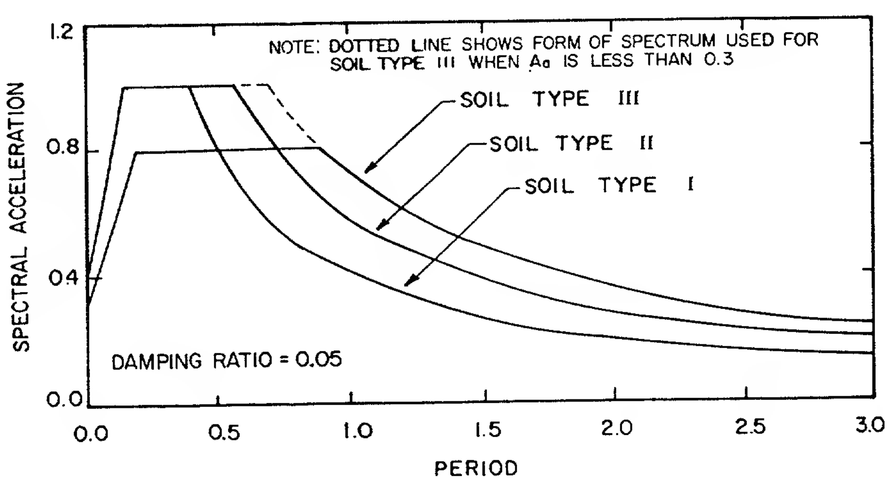


Figure 59: Ground Motion Spectra for A = 0.4 (from Reference 4)

Soil Profile Type III is a profile with soft to medium-stiff clays and sands, characterized by 30 ft. (10 m) or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

Ground motion spectra for 5 percent damping are obtained by multiplying the normalized spectral values shown in figure 58 by the appropriate Acceleration Coefficient obtained from figure 55 and by a correction factor of 0.8 if Soil Profile Type III exists. The resulting ground motion spectra for an Acceleration Coefficient of 0.4 are shown in figure 59. It should be noted that these spectra are modified before they are used in the design provisions (section 6.3).

Ground motion spectra for vertical motions may be determined with sufficient accuracy by multiplying the ordinates of the spectra for horizontal motions by a factor of 0.67.

6.3 DESIGN COEFFICIENTS AND DESIGN SPECTRA

The determination of appropriate seismic design loads, although complex in reality, has been significantly simplified for code application.

For the simplified single degree of freedom model (section 3.1), the lateral earthquake design force (F) is generally express as a fraction on weight (W) of the superstructure. The basis for this expression is as follows:

$$\begin{aligned} \text{force} &= \text{mass x acceleration (Newton's Second Law)} \\ &= Ma \end{aligned}$$

Now, if acceleration is expressed as a fraction of the acceleration due to gravity (g),

$$\begin{aligned} \text{force} &= M.Cg \\ &= C.Mg \\ &= CW \end{aligned} \tag{22A}$$

where C is an acceleration coefficient (fraction of gravity).

In modern design codes, C is called a lateral design force coefficient or seismic load coefficient and is a function of:

- seismic zone.
- period of the bridge.
- site soil conditions.

For use in a design code, it is advantageous to express the lateral design force coefficient in as simple a manner as possible. For example, in the AASHTO Guide Specification [reference 4] this coefficient is identified as C_S, and is given by:

$$C_S = \frac{1.2AS}{T^{2/3}} \tag{22B}$$

where A is an acceleration coefficient as given by the seismic zoning map of figure 55;
S is a site coefficient as given in table 4;

and T is the period of the bridge in seconds.

Note that this is an empirical relationship and T must be expressed in seconds for it to be valid. Coefficients A, S and C_S are nondimensional and therefore are independent of the chosen system of units.

Figure 60 shows C_S plotted against T for various site and acceleration coefficients. Note that the upper limit for C_S is 2.5A. For Soil Profile Type III, in areas where A exceeds 0.30, the maximum value for C_S is 2.0A. Note also that the use of a simple soil factor in the equation directly approximates the effect of local site conditions on the design requirements as discussed in section 6.2.

Table 4: Site Coefficient (S)

	Soil Profile Type		
	I	**II**	**III**
S	1.0	1.2	1.5

In the discussion on spectral shapes in section 6.2 and shown in figures 58 and 59, the recommended ground response spectra decreases approximately as 1/T for longer periods. However, because of the concerns associated with inelastic response of longer period bridges it was decided that the ordinates of the design coefficients and spectra should not decrease as rapidly as 1/T but should be proportional to $1/T^{2/3}$.

A comparison of the spectra resulting from equation (22) and those of the AASHTO Guide Specification recommended ground response spectra, is given in figure 61. It will be seen that in the development of AASHTO Guide Specifications, the elastic seismic response coefficient and spectra is approximately 50 percent greater at a period of 2 sec. (for stiff soil conditions) than would be obtained by direct use of the recommended ground response spectra. This increase gradually decreases as the period of the bridge shortens. The two major reasons for introducing this conservatism into the design coefficients and spectra (for long period bridges) are:

1. The fundamental period of a bridge increases as the column height increases, the span length increases and the number of columns per bent decreases. Hence the longer the period the more likely that high ductility requirements will be concentrated in a few columns.

2. Instability of a bridge is more of a problem as the period increases.

The determination of the design coefficient or design response spectra generally includes the following steps:

STEP 1 Determine the Seismic Zone for the bridge site. This is generally done by reference to a seismic zoning map; e.g., figure 55 for the AASHTO Guide Specifications and figure 56 for the AASHTO Standard Specifications.

STEP 2 Determine the Acceleration Coefficient for the appropriate zone--in the AASHTO Guide Specifications the Acceleration Coefficient is given on the zoning map of figure 55.

In the AASHTO Standard Specifications and current CalTrans requirements, acceleration coefficients are specified for each zone of the map.

STEP 3 Determine the site coefficient for the bridge site--in the AASHTO Guide Specification there are three soil profiles defined and a site coefficient (table 4) is assigned to each profile.

In the AASHTO Standard Specifications and the Caltrans requirements the site coefficient for four soil profiles is incorporated directly in the plots of the response coefficient (AASHTO) and design spectra (Caltrans).

STEP 4 Determine the design coefficient or design response spectra--in the AASHTO Guide Specifications the design coefficient and response spectra are expressed as a function of the acceleration coefficient, site coefficient and period of the bridge. Spectra with varying Acceleration Coefficients and site coefficients are shown in figure 60.

In the AASHTO Standard Specifications requirements, coefficients are presented as plots. There are four separate plots for each of four soil types. The design coefficient plots incorporate a reduction factor for ductility and risk assessment, i.e., these plots have already been reduced assuming ductile performance of the substructures. In the CalTrans requirements, design response spectra are also presented as plots but are **not** reduced. As with AASHTO there are four separate plots for each of four different soil types. The design coefficient is obtained from the design response spectra.

6.4 RESPONSE MODIFICATION FACTORS

In the AASHTO Guide Specifications, seismic design forces for individual components and connections of bridges are determined by dividing the elastic forces obtained from the analysis (using the design coefficient or spectra specified in section 6.3) by the appropriate Response Modification Factor (R). The Response Modification Factors for the various components are given in table 5.

Response modification factors (R-factors) are introduced to implement the design philosophy of the AASHTO Guide Specification as outlined in section 4.1. They are used to obtain the design forces for each component using the results of an analysis of the bridge when subject to the seismic loads of the elastic design spectra. Inherent in the R-factors is the assumption that the columns will yield when subjected to the elastic forces induced by the design ground motions and that connections and foundations are to be designed to accommodate the design ground motion forces with little, if any, damage.

The rationale used in the development of the R-factors for columns, piers and pile bents is based on considerations of redundancy and ductility provided by the various supports. The wall type pier is judged to have minimal ductility capacity and redundancy

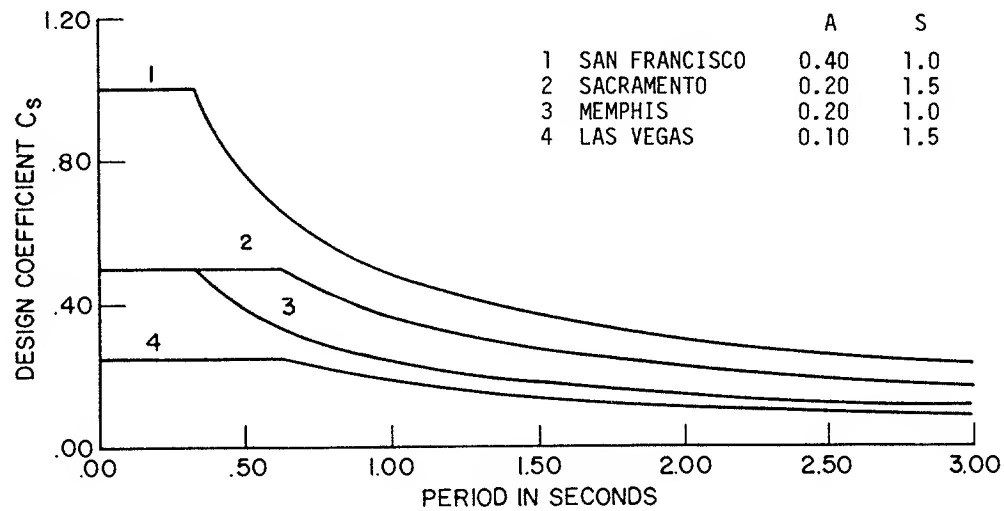


Figure 60: Representative Design Coefficient and Spectra Curves for Four Different Locations

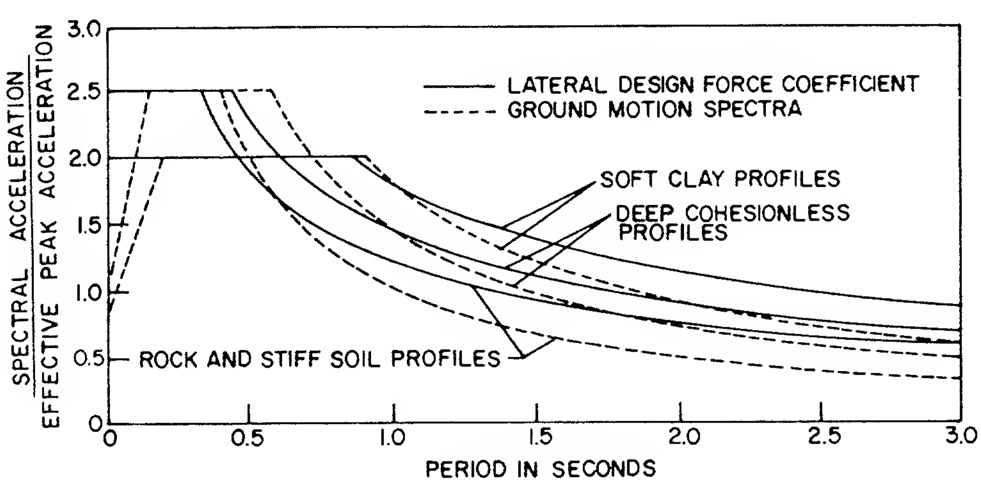


Figure 61: Comparison of Free Field Ground Motion Spectra and Lateral Design Force Coefficients (from Reference 4)

Table 5: Response Modification Factors (R)

Substructure[1]	R	Connections	R
Wall-Type Pier[2]	2	Superstructure to Abutment	0.8
Reinforced Concrete Pile Bents			
a. Vertical Piles Only	3	Expansion Joints Within a Span of the Superstructure	0.8
b. One or more Batter Piles	2		
Single Columns	3		
		Columns, Piers or Pile Bents to Cap Beam or Superstructure[3]	1.0
Steel or Composite Steel and Concrete Pile Bents			
a. Vertical Piles Only	5		
b. One or more Batter Piles	3	Columns or Piers to Foundations	1.0
Multiple Column Bent	5		

[1] The R-Factor is to be used for both orthogonal axes of the substructure.

[2] A wall-type pier may be designed as a column in the weak direction of the pier provided all the provisions for columns in chapter 8 of the AASHTO Guide Specification [reference 4] are followed. The R-Factor for a single column can then be used.

[3] For bridges classified in Seismic Performance Category C and D (see section 6.5 for definition), it is recommended that the connections be designed for the maximum forces capable of being developed by plastic hinging of the column or column bent. These forces will often be significantly less than those obtained using an R-Factor of 1 or 0.8.

in its strong direction and is therefore assigned an R-factor of 2. A multiple column bent with well-detailed ductile columns, is judged to have good ductility capacity and redundancy and is therefore assigned the highest value of 5. Although the capacity of single columns is similar to that of columns in a multiple column bent, there is no redundancy and therefore a lower R-factor of 3 is assigned to these columns. This should then provide a level of performance similar to that of a multiple column bent. At the time these R-factors were compiled, there was little information available on the performance of pile bent substructures. Since that time Caltrans has reported the satisfactory performance of several bridges with pile bents during actual earthquakes. The conservative R-factors in table 5 were based on the judgment of potential pile bent performance in comparison to that of the other three types of substructure. It was believed that there would be a reduction in the ductility capacity of pile bents with batter piles and therefore lower R-factors were assigned to these systems. In the light of the Caltrans experience, there may now be some justification for increasing the factors for these substructures.

The R-factors of 1.0 and 0.8 assigned to connections mean that these components are designed for the design elastic forces and for greater than the design elastic forces in the case of abutments and expansion joints within the superstructure. This approach is adopted in part to accommodate the redistribution of forces that occurs when a bridge responds inelastically and to maintain the overall integrity of the bridge structure at these important joints. Increased protection can be obtained for a minimum increase in construction cost by designing connections for these larger force levels. However, it should be noted that for bridges classified in Seismic Performance Category C and D (see section 6.5) the recommended design forces for column connections are the forces that can be developed by plastic hinging of the columns. Since these are the maximum forces that can be developed and they are generally smaller than the elastic values, the desired integrity will be obtained at lower cost. The connection design forces associated with plastic hinging are not specified for bridges in Seismic Performance Category B because the calculation of these forces requires a more detailed analysis. However they may be used if desired.

6.5 SEISMIC PERFORMANCE CATEGORIES

A basic premise in developing the AASHTO Guide Specifications was that they be applicable to all parts of the United States. The seismic risk varies from very small to high across the country and design requirements applicable to the higher risk areas are not always appropriate for the lower risk areas. In order to provide flexibility in specifying design provisions associated with areas of different seismic risk, four Seismic Performance Categories (SPC) are defined. The four categories permit variation in the requirements for methods of analysis, minimum support lengths, column design details, foundation and abutment design requirements in accordance with the seismic risk associated with a particular bridge location.

Therefore, in these Guide Specifications, each bridge is assigned to one of four Seismic Performance Categories (SPC), A through D, as shown in table 6. This method of classification is based on the Acceleration Coefficient (A) determined from the map shown in figure 55 and discussed in section 6.1, and the Importance Classification (IC), discussed in section 6.6. As noted above, minimum analysis and design requirements are governed by the SPC.

Table 6: Seismic Performance Categories (SPC)

ACCELERATION COEFFICIENT	IMPORTANCE CLASSIFICATION (IC)	
A	I	II
A $\leq$ 0.09	A	A
0.09 < A $\leq$ 0.19	B	B
0.19 < A $\leq$ 0.29	C	C
0.29 < A	D	C

It is seen in table 6 that bridges classified as SPC D are designed for the highest level of seismic performance whereas those classified as SPC A are to be designed for the lowest level of seismic performance.

6.6 IMPORTANCE CLASSIFICATION

In the AASHTO Guide Specifications, an Importance Classification (IC) is assigned to all bridges with an Acceleration Coefficient greater than 0.29 for the purpose of determining the Seismic Performance Category (SPC) in section 6.5. Bridges are classified as either essential or non-essential on the basis of Social/Survival and Security/Defense requirements. In general, an essential bridge is one that must continue to function after an earthquake and is assigned to group I for its Importance Classification, IC. For all other bridges, IC = II.

The determination of the Importance Classification for a bridge is necessarily subjective and in addition to the Social/Survival and Security/Defense requirements, consideration should also be given to average annual daily traffic.

The Social/Survival evaluation is largely concerned with the need for roadways during the period immediately following an earthquake. In order for civil defense, police, fire department or public health agencies to respond to a disaster situation a continuous route must be provided. Bridges on such routes should be classified as essential. In addition, any bridge that crosses an essential route should also be classified as essential. This is because of the risk of closure to the essential route by the collapse of the overcrossing.

A basis for the Security/Defense evaluation is the 1973 Federal-Aid Highway Act which required that a plan for defense highways be developed by each State. This plan had to include, as a minimum, the interstate and Federal-Aid primary routes. However, some of these routes can be deleted when such action is considered appropriate by a State. The defense highway network provides connecting routes to important military installations, industries and resources not covered by the Federal-Aid primary routes. Bridges serve as important links in the Security/Defense roadway network and such bridges should also be classified as essential.

CHAPTER 7 DESIGN FORCES AND DISPLACEMENTS

This chapter examines appropriate methods of calculating forces and displacements under the action of the design loadings. This is achieved through structural analysis, which is in essence an art rather than a science. The guidelines presented in this chapter represent minimum requirements, and should be critically examined for applicability on a case by case basis. Any attempt to rigidly define analytical requirements for all future projects understates the "art" content of analysis and curtails independent and creative thinking. The following guidelines should be interpreted in this light.

7.1 ANALYSIS PROCEDURES

Many of the analysis methods described in chapter 3 can be performed manually using hand procedures or they may be programmed for a computer based solution. This latter option is preferable for all but the simplest bridges because even the most primitive analysis methods become tedious once a bridge becomes even slightly complex. For example, bridges which are continuous for more than two spans or those with multi-column bents and flexible bent caps on soft foundations will be time consuming to analyze by hand. Since most bridge design offices have access to computer facilities, it is expected that engineers routinely engaged in bridge design will prefer to use computer programs for analysis. Therefore, the basic thrust of this section is towards computer-based analysis procedures.

Analytical procedures have advanced considerably, to the point where elastic dynamic analysis is being used routinely in areas of moderate to high seismic risk. It is often thought that dynamic (or "rigorous") analysis allows the prediction of earthquake forces and displacements very accurately. This assumption is incorrect. At best, a dynamic analysis of a well modelled structure will give the engineer a good indication of the distribution of forces in the structure and the magnitude of the deformations to be expected. Thus it is unnecessary to overly refine a dynamic analysis once "reasonable" results are obtained.

Structural analysis by computer is a sophisticated design tool but it should only be used when the assumptions made by the author of the computer program are understood. Analytical results should always be examined critically to check that basic criteria, such as equilibrium, have been satisfied. The engineer must retain a "feel" for the structure, and be able to predict and check the global response independently of the computer analysis.

7.1.1 Applicability of Various Methods

Recommendations for choice of analysis method are usually given in terms of the type of bridge (number of spans, geometric configuration and regularity) and its Seismic

Performance Category (see section 6.5). These recommendations are summarized in table 7. In this table "S" refers to a single mode analysis while "M" refers to a multimode analysis (section 3.1).

Table 7: Analysis Method Recommendations

Seismic Performance Category	Regular Bridges with 2 or More Spans	Irregular Bridges with 2 or More Spans
A	-	-
B	S	S
C	S	M
D	S	M

In this table, the terms **regular** and **irregular** are used. A **regular** bridge is defined as one having no abrupt or unusual changes in mass, stiffness or geometry (subtends a plan angle of no greater than 90 degrees) along its length, and no large differences (greater than 25 percent of the lesser of the two quantities) in these quantities between adjacent supports (abutments excluded). An **irregular** bridge is one that does not satisfy the definition of a regular bridge. Examples of regular and irregular bridges are illustrated in figure 62.

Time history analysis may be required in special circumstances where a travelling wave may cause out-of-phase motions at the piers of a very long structure.

7.1.2 Single Mode Analysis

As noted above, the single mode method is recommended for the analysis of "regular" bridges instead of the more rigorous multimode method. Even "irregular" bridges in seismic performance category B, may be analyzed by this method. This is done to avoid the need for computer based modal solutions for simple bridges, or for those in low seismic risk areas. In many cases, the method gives perfectly satisfactory results but it should be noted that various approximations are made in order to apply the method to these "regular" bridges. These approximations introduce uncertainties in the calculated results especially for bridges which are almost "irregular" but classified as "regular" under the above rules (table 7). Therefore, if modal analysis capability is available, it is recommended that it be used instead of the single mode approach, even for regular bridges.

The reason that the single mode method should only be applied to regular bridges is because the method assumes that **one, time-independent, shape** with time-varying amplitude completely defines the earthquake motion of the structure. In other words, these regular bridges are assumed to respond to earthquake loads in a single mode of deformation which retains its shape throughout the duration of an earthquake. However, the size of this deformation mode (i.e. the amplitude of vibration) may change with time in accordance with the variations in ground motion. This is a reasonable assumption for regular, uniform structures, but may be in gross error for more complex

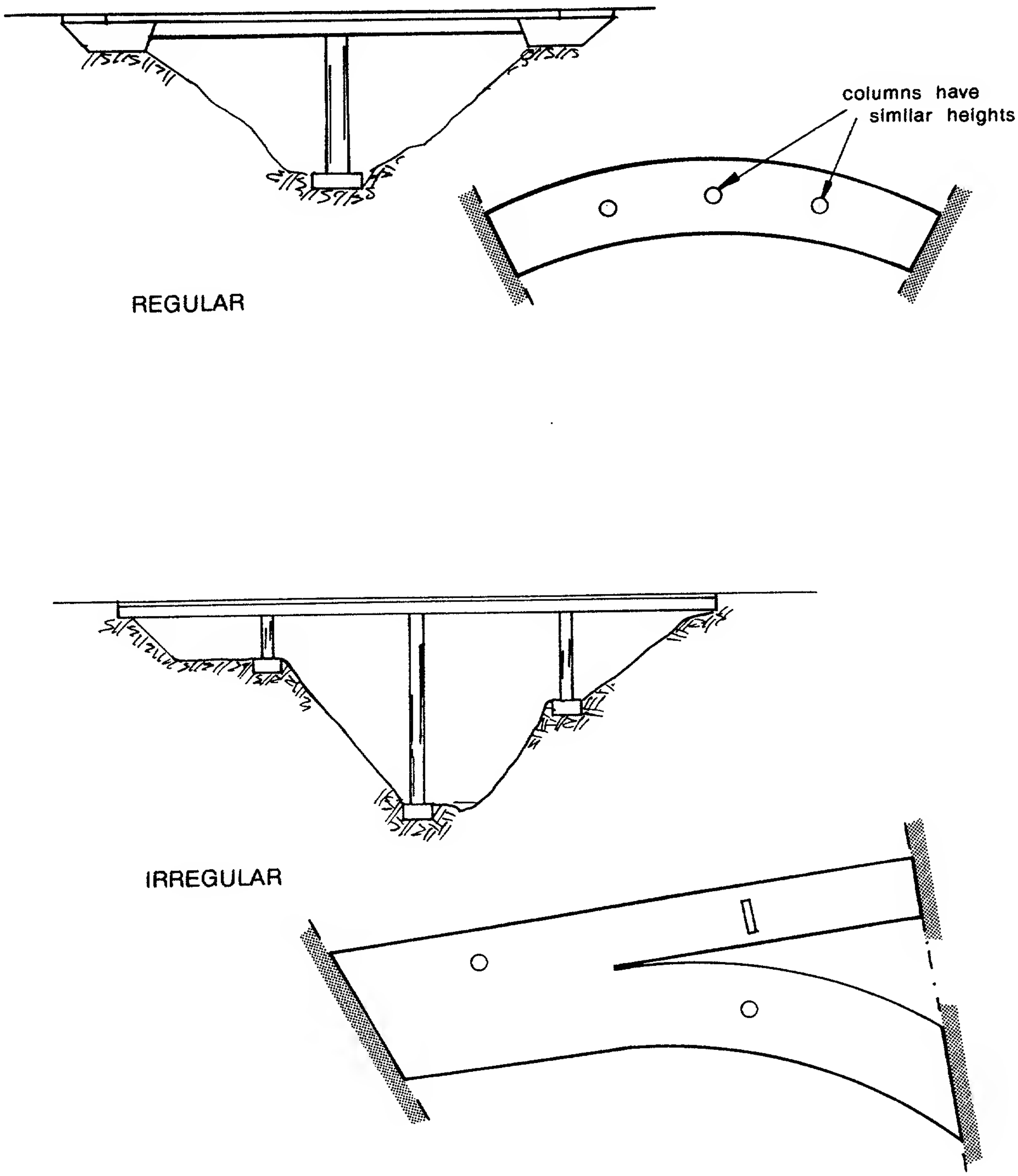


Figure 62: Regular and Irregular Bridges

bridges. The single mode analysis is applied independently in the longitudinal and transverse directions, and the results are combined as described in section 7.3.

The single mode analysis is based upon the first steps of a Rayleigh analysis. This analysis assumes a vibration shape for a structure, calculates maximum potential and kinetic energies associated with that shape, and by equating these two energies deduces a natural frequency for the system. The design forces and displacements for the system come from a static analysis using the inertia forces consistent with the initial vibration shape as the applied loading. This is the essence of the single mode analysis, which formulates an equivalent model of the system, such that the overall behavior of the complete bridge is closely approximated. Review section 3.1 for earlier discussion on bridge modelling.

In the description of the single mode method that follows, a change in notation is introduced. This is done so that this presentation is consistent with that in the AASHTO Guide Specification [reference 4]. Unfortunately, the simplified notation used in chapter 3 is not universally adopted and when writing the following material, uniformity with the AASHTO Guide was considered preferable to uniformity with chapter 3. Table 8 summarizes the relevant changes in notation.

Table 8: Notation Changes

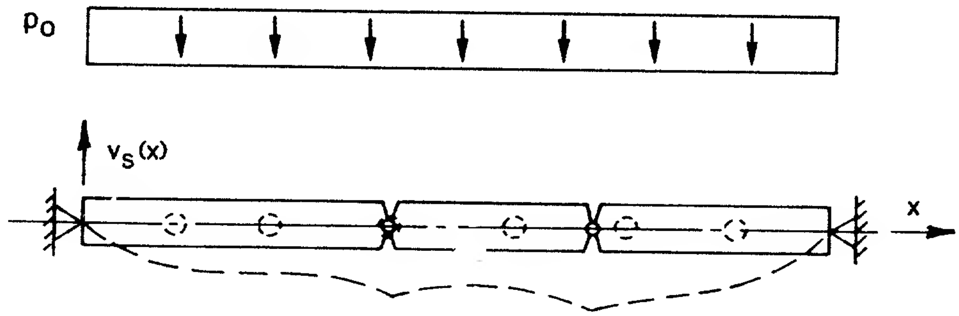
Chapter 7 Notation	Description	Chapter 3 Notation
p_o, p, p_e	lateral loads	f
v_s, v_{max}	displacement	d
ω	bridge frequency	p
ξ	damping ratio	n

The weight per unit length of the structure is given by w(x), and the mass per unit length is given by m(x). The initial assumed vibration shape is the static deflection, $v_s(x)$, due to a unit (p_o = 1) uniform load acting in the direction of analysis (see figure 63a). This may be calculated by hand or with the aid of a computer.

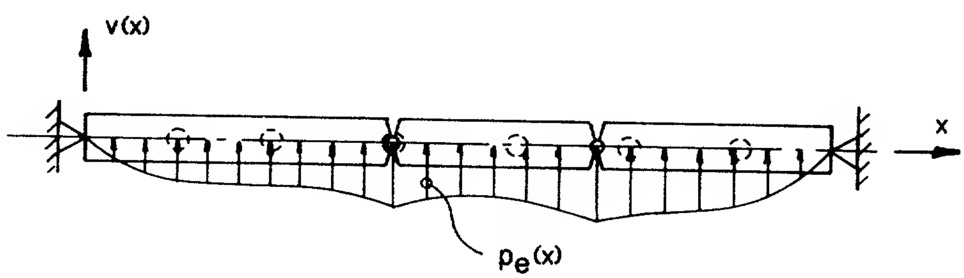
The strain (potential) energy, U, stored in the structure in this deflected position is equal to the work, W_E, done by the external load in deforming the structure:

$$U = W_E = \frac{1}{2} p_o \alpha \qquad \text{where} \quad \alpha = \int_0^L v_s(x) dx \tag{23}$$

If the structure is now released from its deflected position, it will vibrate **in the same shape** with frequency ω. The maximum kinetic energy, T_{max}, occurs when the structure passes through its at-rest position, and is given by:



(a) Deflected Shape due to Uniform Static Loading



(b) Characteristic Static Loading Applied to the Bridge System

Figure 63: Single Mode Method of Analysis
(after Reference 4)

$$T_{max} = \frac{\omega^2}{2g}\gamma \qquad \text{where } \gamma = \int_0^L w(x)[v_s(x)]^2 dx \tag{24}$$

Equating these two energies, and recognizing that $\omega = 2\pi/T$, an expression for the period of vibration is obtained as follows:

$$p_o\alpha = \frac{\omega^2}{g}\gamma \qquad T = 2\pi\sqrt{\frac{\gamma}{p_o g\alpha}} \tag{25}$$

At first sight, equation 25 does not look like the familiar expression for period as developed in figure 9a. However, if γ/g is thought of as an effective mass and $p_o\alpha$ as the same as an effective stiffness, equation 25 takes on the appearance of the earlier formula. The fact that γ/g and $p_o\alpha$ are equivalent to mass and stiffness respectively can be demonstrated by considering the longitudinal response of a straight, uniform ($w(x) = w$) bridge for which $v_s(x)$ is a constant (Δ) and the integrations in equations 23 and 24 for α and γ both reduce to the constant L.

Then	γ	$= w.\Delta^2.L = W.\Delta^2$	(25a)
and	α	$= \Delta L$	(25b)
therefore	$p_o\alpha$	$= p_o\Delta L = P_o\Delta = K_o\Delta^2$	(25c)
where	W	$= w.L$ ... total weight	(25d)
	P_o	$= p_o.L$ ... total lateral load	(25e)
	K_o	$= P_o/\Delta$ ... longitudinal stiffness	(25f)

Substitution into equation 25 and simplifying gives

$$T = 2\pi\sqrt{\frac{W}{gK_o}} \tag{25g}$$

as before (figure 9a).

Alternatively, this equivalence may be numerically demonstrated using the examples in Chapter 8 and 10.

The period allows the appropriate amplitude for the assumed shape to be determined from the ordinate of the response spectrum, $S_a(\xi,T)$. A damping ratio of 5 percent is typically assumed. This response is calculated as a scalar times the initial deflected shape, $v_s(x)$, due to the uniform load.

maximum acceleration $= S_a\ (\xi,T) = C_s g$

maximum displacement $= S_a\ /\ \omega^2 = C_s g\ /\ \omega^2$

deflected shape at time
of maximum displacement $= v_{max}(x) = \frac{C_s g}{\omega^2}\frac{\beta}{\gamma}v_s(x)$

$$\text{where} \quad \beta = \int_0^L w(x)v_s(x)dx \tag{26}$$

The $(\beta\ /\ \gamma)$ factor can be thought of as a normalizing factor since the magnitude

of $v_s(x)$ was chosen arbitrarily (i.e., corresponding to a unit uniform load).

The design forces and displacements could be calculated by simply multiplying those corresponding to the initial shape by the above scalar $(C_s g\beta \ / \ \omega^2\gamma)$. However, more accurate forces and displacements are obtained by taking the analysis a little further. The inertia forces, $p_e(x)$, corresponding to the deflected shape at the time of the maximum displacement are:

$$p_e(x) = k^* v_{max}(x) \tag{27a}$$

where k^* is an equivalent stiffness. Then, from the expression for frequency (figure 9a)

$$\omega^2 = k^*/m(x) \tag{27b}$$

and substitution into the above expression for $v_{max}(x)$ gives:

$$v_{max}(x) = \frac{C_s \cdot g}{k^*} \cdot \frac{\beta}{\gamma} \cdot v_s(x) \cdot m(x) \tag{27c}$$

Now substitution of equation (27c) into (27a) and noting that

$$w(x) = m(x).g$$

gives

$$p_e(x) = C_s \cdot \frac{\beta}{\gamma} \cdot w(x) \cdot v_s(x) \tag{27d}$$

These forces are applied to the structure (see figure 63b) to obtain the member forces and displacements for use in design. Again, this can be performed by hand calculation or with the aid of a computer.

It should be apparent that this process is the beginning of an iterative scheme, and that the previous calculations could be repeated for this new load pattern, leading to a new estimate of the structural period and the subsequent response. The process converges when the deflected shape corresponding to the inertia forces is in agreement with the assumed shape at the beginning of the iteration. At convergence, the deflected shape and the fundamental vibration shape correspond. However, this extra effort is unwarranted, because for regular structures the deflected shape corresponding to a uniform load will closely approximate the mode shape, and the period and response calculated above will be good estimates of the actual reponse.

In summary, the analysis is performed in both the longitudinal and transverse directions, and the steps for each direction are as follows:

- Calculate $v_s(x)$.
- Calculate α, β, γ.
- Calculate T and C_s.
- Determine $p_e(x)$.
- Calculate the design forces and displacements.

A more detailed explanation of the method is given in the AASHTO Guide Specification [reference 4]. Also, as noted above, numerical examples which illustrate the method are presented in chapters 8 and 10.

7.1.3 Multi-Mode Analysis

Multi-mode spectral analysis should be used for bridges with irregular geometry, mass or stiffness properties. Irregularities generally induce coupling between responses in the three global directions, making the assumption of a single mode for each of the longitudinal and transverse directions inappropriate. In addition, the total response of an irregular structure is not dominated by just one mode of vibration, but rather, several modes contribute significantly, and must each be included for meaningful results to be obtained. A computer program should be used by an experienced analyst to account for these effects accurately. Currently available suitable programs are briefly described in section 7.8.

Sufficient modes should be included in the analysis to ensure that the effective mass included in the model is at least 90 percent of the total mass of the structure. (The total effective mass should be printed in the output from the computer analysis. A discussion of effective mass can be found in any basic dynamics text such as reference 17. If this requirement is not satisfied, missing mass corrections should be made. The mass and stiffness of the entire seismic resisting system should be included in the analysis.

A multi-mode spectral analysis calculates the maximum response of the structure in each of the modes of vibration included in the analysis. These maxima are then combined to give the total response of the structure. Care must be taken with this combination, for these modal maxima do not generally occur at the same instant in time. The generally accepted modal combination rule is the Square Root of the Sum of the Squares (SRSS) method. This method works well for structures with well separated natural periods, and should be adequate for most bridge structures. However, when closely spaced modes occur, more sophisticated combination rules should be used.

Perhaps the most elegant of these rules is the Complete Quadratic Combination (CQC) rule, which accounts for statistical correlation between the various modal responses. Other rules for closely spaced modal combination include absolute summation of responses from those modes with frequencies within 10 percent of the lowest frequency in the group, and then SRSS combination of the remaining, well separated, modes. It should be pointed out that absolute summation has the potential to grossly overestimate response in directions orthogonal to the input and in situations where modal contributions tend to cancel one another. For a discussion of these various combination methods, see reference 33.

7.1.4 Time History Analysis

Time history analysis should be used for very unusual structures, and especially for very long structures where travelling wave effects can invalidate the response spectrum assumption that all support points have identical motions.

Time history analysis requires a detailed description of the time variation of the ground acceleration at all support points for the structure. It is impossible to describe these variations in such a way as to cover future motions likely to occur at the site. However,

this problem can be addressed by using several time histories, each of which has the overall characteristics of the design spectrum, but each one having different and potentially important characteristics in terms of details of the structural response.

One of the key parameters in a time history analysis is the time step. This is specified so as to accurately capture the response of all significant modes, and as a rule of thumb, this should be set at one-tenth of the period of the highest mode of interest.

Unlike response spectrum analysis, the time variation of all response quantities is explicitly computed, and combination of modal maxima is not an issue.

7.2 PRACTICAL MODELLING GUIDELINES

The type and degree of refinement of the mathematical model depends on the complexity of the bridge under consideration and the amount of detail required of the results. The overall objective should be to produce a model that will capture the essential dynamic characteristics of the bridge and produce realistic overall results. The designer must be able to reconcile the results from the computer analysis as "making sense", and no amount of analysis can replace the need for careful design and thoughtful detailing of the actual structure. This section is intended to provide some basic guidelines which, when followed, will produce reasonable results for most structures.

7.2.1 Structural Geometry

The selection of the nodal locations for the model determines the accuracy with which the model can respond to the applied loads. The superstructure should, as a minimum, be modelled as a series of three dimensional beam-column members with nodal locations at the ends of each span and at the quarter points within each span. Discontinuities and joints in the superstructure should be explicitly modelled. This may be achieved by the use of "double nodes" at expansion joints, coupled by elements representing the stiffness of the joint. Significant horizontal and vertical curvatures should be accurately modelled, as should skew supports.

Intermediate columns or piers should also be modelled as three dimensional beam-column members. Generally, for short, stiff columns with lengths less than one-third of either of the adjacent span lengths, intermediate column nodes are unnecessary. However, for long, flexible columns, intermediate nodes at the third points should be used. The model should consider the eccentricity of the columns with respect to the superstructure. In other words, a rigid zone at the tops of the column elements should be used to model the vertical distance between the soffit of the bridge superstructure and its geometric centerline. It may also be necessary to include rigid zones in the superstructure members to accurately represent the clear span. Each zone would then model the horizontal distance between the column centerline and the column face.

Additional nodal points may be required at the pier locations below ground level to model the effects of foundation flexibility. This is described in detail in section 7.2.4. figure 71 illustrates nodal locations for two different abutment configurations, which are also discussed in section 7.2.4.

7.2.2 Mass Distribution

The structural mass should be lumped in such a way as to capture all significant reponse patterns. Typically, three mass nodes within each span should suffice. Each node in a typical model will have six degrees of freedom, three translations and three rotations. No rotational mass need be specified.

The mass should take into account the superstructure, pier caps, abutments, columns, and foundations. It should also include the effects of roadway surfacing, utilities, crash barriers and the like. Generally, the inertial effect of live load is ignored. However, it may be appropriate to include some live load inertial effects in those short-span bridges which have high live to dead load ratios.

7.2.3 Material and Section Properties

Standard material properties should be used in all cases. It is especially important to accurately reflect the material properties of the more flexible elements in the bridge, such as rubber bearings and soft soils. Properties for rubber bearings should be taken from product catalogs or test data when available.

Soil stiffnesses should be computed in collaboration with a competent geotechnical engineer, especially for sites with "soft" soil conditions. In addition, guidance on selection of these stiffnesses can be obtained from the recent state-of-the-art report on highway bridge foundation design and analysis for earthquake loads [reference 6].

Where the surface materials are very soft, the effective ground line should be taken at a depth underlying such materials. When a great depth of very soft material exists, special investigation of stiffness and strength properties should be undertaken. Such soils are not normally suitable for bridge foundations in seismic areas. For soft sites with uncertain stiffness parameters, it is recommended that analyses using upper and lower bound soil stiffnesses be performed to determine the sensitivity of response to the soil conditions. Suggested soil stiffness parameters for use in preliminary seismic analyses are given in table 9.

Where preliminary analyses have indicated that the seismic response of the bridge is significantly affected by the soil stiffness, the final analysis should use a set of soil stiffnesses compatible with the foundation displacements at the design loading.

It is important to accurately model the in-plane lateral stiffness and torsional stiffness of the superstructure, particularly for the transverse analysis of a continuous bridge. These stiffness parameters influence the distribution and magnitude of lateral loads to which the substructure will be subject. For "stiff" superstructures (e.g. box sections), the in-plane lateral stiffness may be based on beam theory using the full width of the section. For more flexible superstructures (e.g. double T sections), the effective in-plane lateral stiffness may be much less than that based on the above assumption. Careful consideration of this stiffness is warranted.

For reinforced concrete sections, the use of cracked or uncracked section properties must be decided. If the section is assumed to be cracked, the member (column) will be more flexible than if uncracked. The bridge will then respond with a longer period and lower forces will be predicted for the column. However, because of the additional flexibility, higher deflections will be calculated even though the forces are

Table 9 : Suggested Soil Stiffness Parameters for Preliminary Seismic Analysis (after Reference 29)

SOIL TYPE	SITE DATA			DESIGN PARAMETERS			
	N (blows/ft.)	Undrained Shear Strength (ksf)	Φ' (degrees)	n_h (kips/ft^3) Dry	n_h (kips/ft^3) Submerged	K_h (kip/ft^2)	E_s (kip/ft^2)
Cohesionless Soils							
- dense	30-50		45	100	60		
- loose	4-10		30	15	6		
Cohesive Soils							
- hard	20-60	3-15				375	520
- medium	8-15	1-2				125	170
- soft	2- 4	0.3-0.6				30	40

N = standard penetration test resistance

K_h = modulus of horizontal subgrade reaction

n_h = constant of horizontal subgrade reaction

$$= \frac{dK_h}{d_z}$$

E_s = soil modulus of elasticity

Φ' = effective soil internal angle of friction

less. When in doubt as to which approach is correct, the analysis should be performed twice using first the cracked and then the uncracked properties and thereby bounding the response. Some guidance on the effective properties of bridge superstructures can be found in reference 34. Based on ambient vibration field testing of a large number of concrete bridges in California, the following moments of inertia are recommended.

For reinforced concrete superstructures:

moment of inertia in torsion	: use 100 percent of gross (uncracked) value
moment of inertia in flexure about horizontal axis	: use 40 to 60 percent of gross value
moment of inertia in flexure about vertical axis	: use 60 to 80 percent of gross value

For prestressed concrete superstructures:

moment of inertia in torsion	: use 200 percent of gross (uncracked) value
moment of inertia in flexure about horizontal axis	: use 120 to 140 percent of gross value
moment of inertia in flexure about vertical axis	: use 100 to 120 percent of gross value

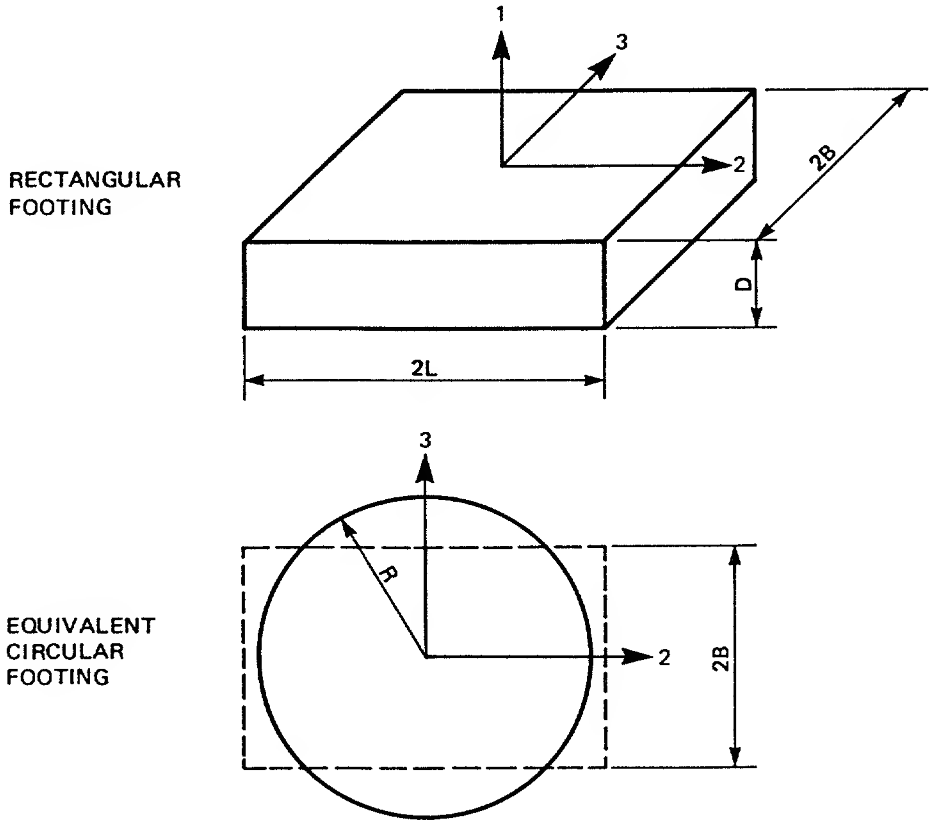
7.2.4 Foundation Modelling

For realistic results from the computer model, the properties of the foundation structure, abutments and supporting soils must also be included. Soil-structure interaction, or foundation compliance as it is sometimes called, can dominate the seismic response of a bridge and must be included. This is particularly true for short, stiff bridges, but is less important for long, tall structures. Ideally the structural model should represent the total system--superstructure, substructure, foundation structures and soil--so that the interaction between soil and structure is captured. However, such an ideal is not presently feasible for the state-of-the-art is not yet advanced to the point where soil properties and interaction models are easily defined. Although several very sophisticated, computer based models have been developed for soil-structure interaction, they are at this time research tools and not suitable for routine design office use. Instead, equivalent but approximate models are in common use to simplify the problem and make it more manageable.

Several of these approximations are summarized below. More detailed descriptions are given in references 4, 6, 29 and 35.

7.2.4A Footings

The most common method for modelling a footing (and other substructures) is to use equivalent springs to represent soil stiffness. In general, the 6 components of displacement (3 translational and 3 rotational) require 6 equivalent spring constants. Rigorously, however there is coupling or interaction between these displacement degrees-of-freedom (especially between the horizontal translation and rotational components) but this coupling is negligibly small for shallow footings. It is also difficult to quantify and for typical highway bridge footings it is usual to neglect these terms. However, if the embedment depth should exceed five times



EQUIVALENT RADIUS:

TRANSLATIONAL: $R_o = \sqrt{\dfrac{4BL}{\pi}}$

ROTATIONAL: $R_1 = \left[\dfrac{4BL\,(4B^2 + 4L^2)}{6\pi}\right]^{1/4}$ (x-AXIS ROCKING)

$R_2 = \left[\dfrac{(2B)^3\,(2L)}{3\pi}\right]^{1/4}$ (y-AXIS ROCKING)

$R_3 = \left[\dfrac{(2B)\,(2L)^3}{3\pi}\right]^{1/4}$ (z-AXIS TORSION)

Figure 64: Equivalent Radii for Rectangular Footings (from Reference 6)

the footing dimension (usually its equivalent diameter), a more refined spring model should be used. See, for example, reference 6. In this case a 6x6 stiffness matrix is generated, representing the complete set of foundation constants which is then input to the computer program being used to model the superstructure.

Spring constants for shallow rectangular footings are obtained by modifying the solution for a circular footing, bonded to the surface of an elastic half-space:

i.e. $K = \alpha\beta K_0$ (28)

where α is the foundation shape correction factor
β is the embedment factor
and K_0 is the stiffness coefficient for the equivalent circular footing (table 10)

To use equation 28, the radius of an equivalent circular footing is first calculated according to the degree-of-freedom being considered. Figure 64 summarizes the appropriate radii. K_0 is then calculated using table 10.

Table 10: Stiffness Coefficients for a Circular Surface Footing

Displacement Degree-of-Freedom	K_0
vertical translation	$4GR/1-\nu$
horizontal translation	$8GR/2-\nu$
torsional rotation	$16GR^3/3$
rocking rotation	$8GR^3/3(1-\nu)$

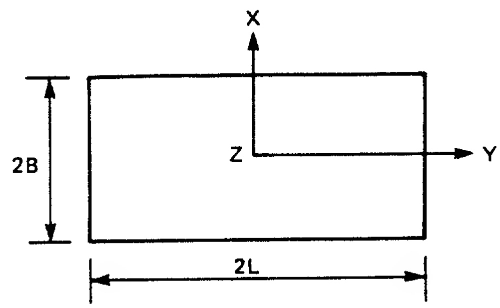
NOTE: G and ν are the shear modulus and Poisson ratio for elastic half space material; R is the radius of the footing.

The shape modification factor, α may be found from figure 65; and the embedment factor, β, from figure 66. Figure 66 is the result of a sensitivity study [reference 6] on typical highway bridge foundations and it will be noted from this figure that β is independent of the actual depth of overburden. This is judged to be a reasonable approximation up to depths of five times the equivalent diameter (2R) at which stage, a special study will be necessary to determine the appropriate stiffness matrix.

7.2.4B Piles

Several possibilities exist for including the effects of piles and surrounding soil into the structural model for seismic analysis. Three of these methods are summarized in figure 67 and include:

- equivalent cantilever model.
- equivalent base springs model.
- equivalent soil springs model.



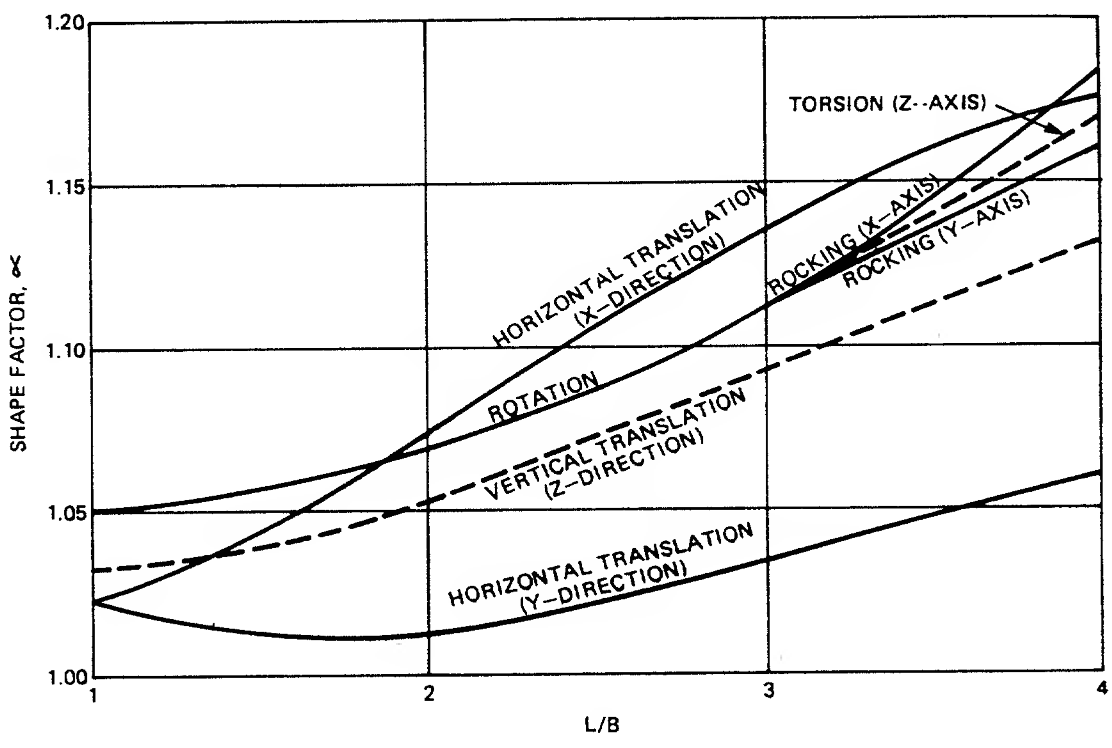


Figure 65: Shape Factors for Rectangular Footings (from Reference 6)

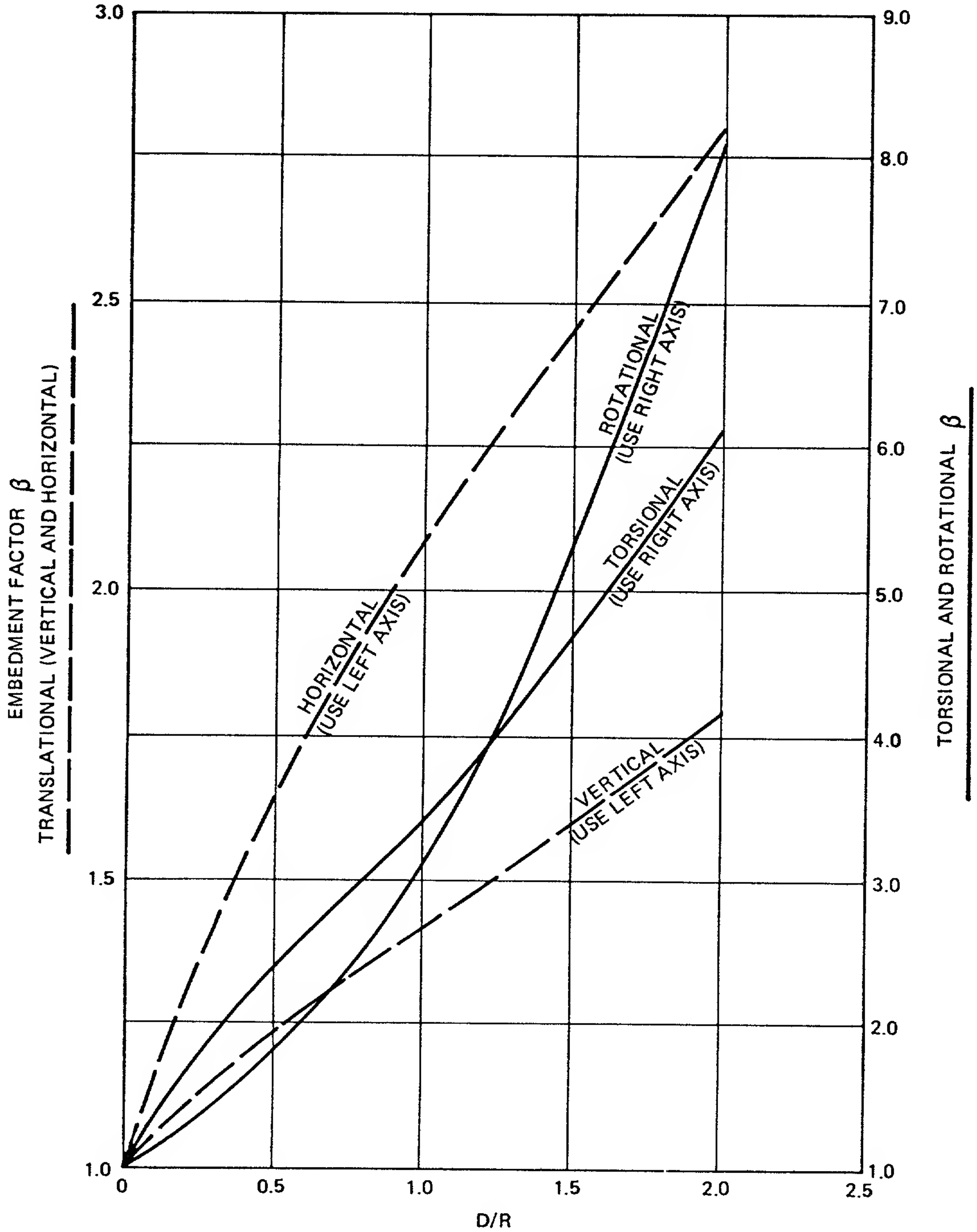


Figure 66: Embedment Factors for Footings (from Reference 6)

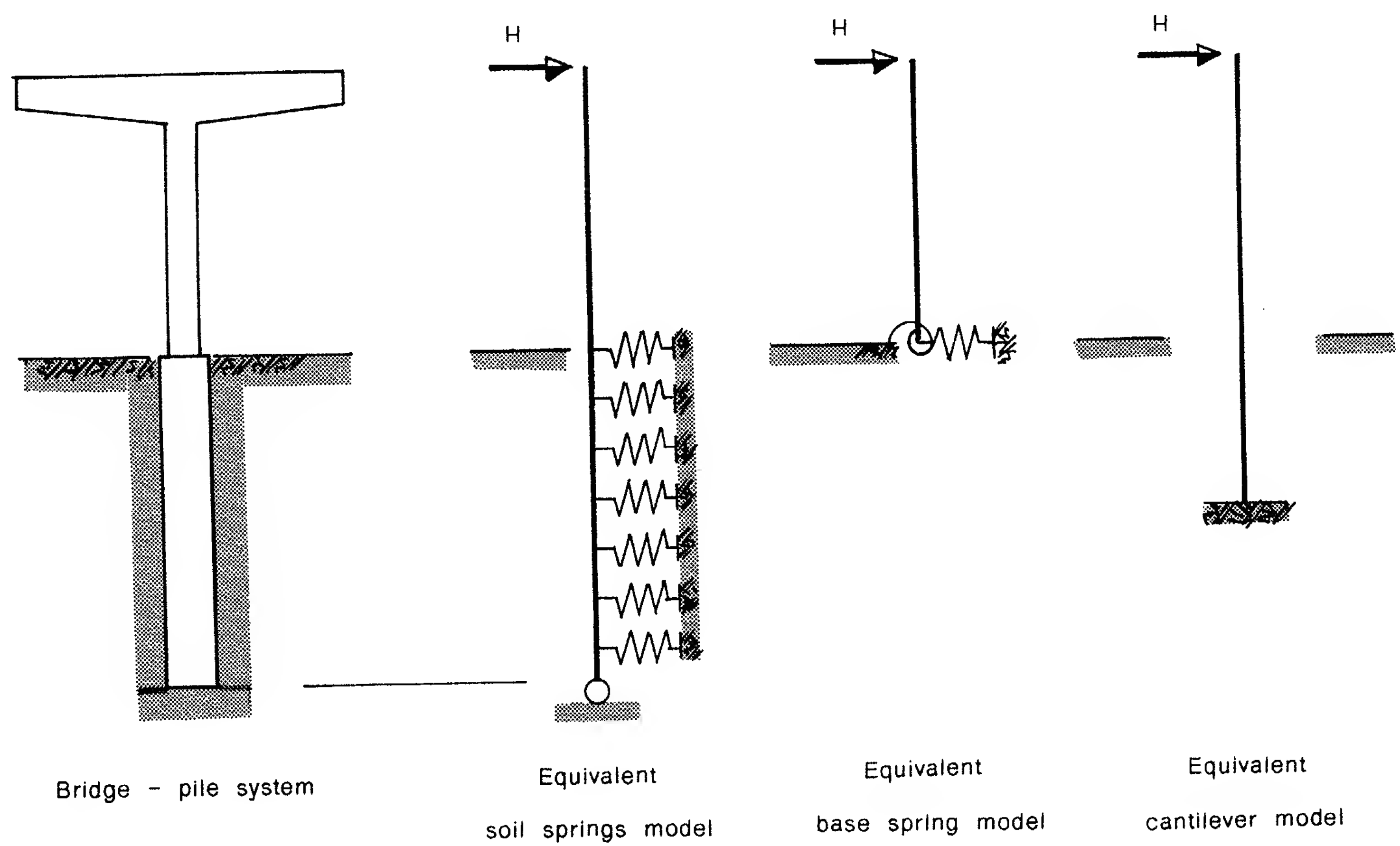


Figure 67: Methods of Representing Pile Foundation Stiffness

The simplest approach is to assume that an **equivalent cantilever column** can be used to model the pile. The section of the cantilever is the same as that of the pile but its length (depth to "fixity") is adjusted so as to give either the same stiffness at ground level or the same maximum bending moment as in the actual soil-pile system.

The length to fixity of the equivalent cantilever can be determined from a detailed substructure model as suggested below, from charts such as those in figures 68a and 68b (which are for large diameter concrete piles (reference 35), or from considerations of the relative stiffnesses of the pile and soil. Using beam-on-elastic-foundation theory, it is possible to show that the equivalent length to fixity is a function of the pile-soil stiffness ratios indicated in figure 69. Note that this formulation gives **two** effective lengths, one for stiffness considerations and the other for maximum pile moments.

In most cases, the use of either the charts or the relative stiffness formulation will give satisfactory results, eliminating the need for a detailed foundation model. Note that the charts give only the effective depth for stiffness considerations, and pile moments based on this length will be overestimated. It should also be noted that the two methods (charts, relative stiffnesses) give different results for the effective depth to fixity. This is in part a reflection of the uncertainty associated with foundation engineering. However, both methods provide a rational and simple way for including foundation flexibility in the seismic analysis of bridges, and results using either method will be closer to the actual behavior than will results from a model which rigidly fixes the bridge at ground level.

Typical ranges for the effective length to fixity (for stiffness) are from 3 to 9 pile diameters, the low end of the range being for very stiff sites. It should be noted that this depth to fixity is potentially a function of the direction of loading, as pile group effects may be different longitudinally and transversely. In the absence of more specific information, the effective modulus of horizontal subgrade reaction (K_h) for each pile may be assumed to vary linearly from 25 percent of the K_h value for a single pile, when the spacing in the direction of load is 3 pile diameters, to the K_h value for a single pile, when the spacing is 8 pile diameters.

The **equivalent base springs** model assumes elastic soil behavior and a set of six independent springs acting at the ground surface. This technique can be quite satisfactory provided the cross coupling terms, which are ignored for footings, are included in the stiffness matrix. However, calculating these terms can be a major effort as it is frequently done by a substructuring technique. That is, a single pile or pile group is modelled explicitly in the soil mass independently of the superstructure, using elastic springs distributed through the depth. Unit displacements are then imposed in turn at the pile cap and the forces necessary to hold these displacements are calculated. These forces are the required stiffness coefficients and will automatically include the cross coupling terms. The method requires the knowledge of the soil spring force-deformation relationships (i.e. the so-called p-y curves) at several points along the pile length. Methods for establishing both linear and nonlinear curves for cohesive and cohesionless soils are available in the literature and have been summarized in reference 6. If linear conditions can be assumed, tables of solutions for combined lateral load and applied moment loading at the pile cap are available in reference 6, from which equivalent spring

constants can be deduced. If large displacements are expected (more than 0.5 inches), linear methods may be inappropriate and a nonlinear model may have to be used. Computer-based solutions will then be necessary and there are several computer codes available for this purpose.

The third technique noted above involves the inclusion of the pile(s) into the superstructure models and the use of p-y curves to represent the soil. This is the **equivalent soil springs** model. The advantage of this approach is the avoidance of the need to calculate equivalent spring constants as in the above method. The disadvantage is the substantial increase in the size of the structural model and the consequential increased demand on computer solution time. Since accuracy is primarily a function of the spacing between nodes used to attach the soil springs to the pile (the closer the spacing, the better the accuracy), and is not so dependent on the pile itself, simple beam column elements are usually adequate for modelling the pile behavior. However, each additional node has a corresponding implication on core storage requirements and solution time as just noted.

Computer models which use soil springs attached at discrete intervals along the length of the pile are similar in concept to analytical solutions based on beam-on-elastic-foundation theory. Another advantage of this model is that it enables a site with layered soils to be represented explicitly. Such a detailed model may be warranted for layered sites with rapid changes in soil stiffnesses.

Until very recently, the **axial stiffness** of the soil pile system was not considered to be important during seismic (lateral) loading. However recent field testing [reference 37] has demonstrated the influence of the rotational stiffness of the foundation structures on seismic response. Where these structures comprise groups of piles, the axial stiffness of each pile contributes significantly to the rotational stiffness of the group.

The various contributions to axial performance are illustrated in figure 70. These are the axial stiffness of the pile itself (EA/L), the shear transfer mechanism (t-z curve) along the sides of pile and the load transfer at the pile tip (Q-z curve). The fundamental problem in determining axial stiffness is quantifying these load transfer relationships. Some guidance is again given in reference 6. The following expressions are taken from this reference, but it is noted that there is no uniform agreement among geotechnical practitioners on the precise form of these relationships.

Side Friction:

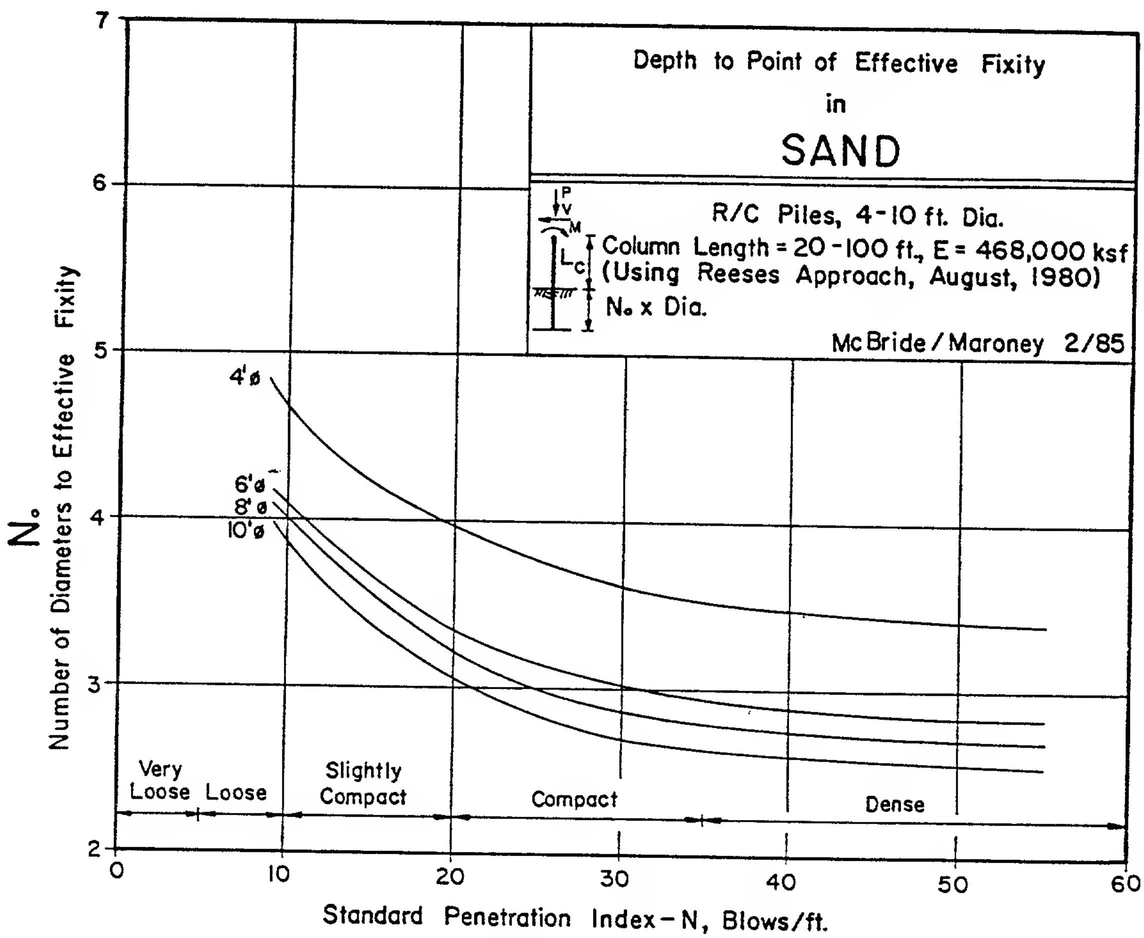
$$f = f_{max} \left(2\sqrt{z/z_c} - z/z_c\right) \tag{29}$$

where

f = unit friction mobilized along a pile segment at movement, z,

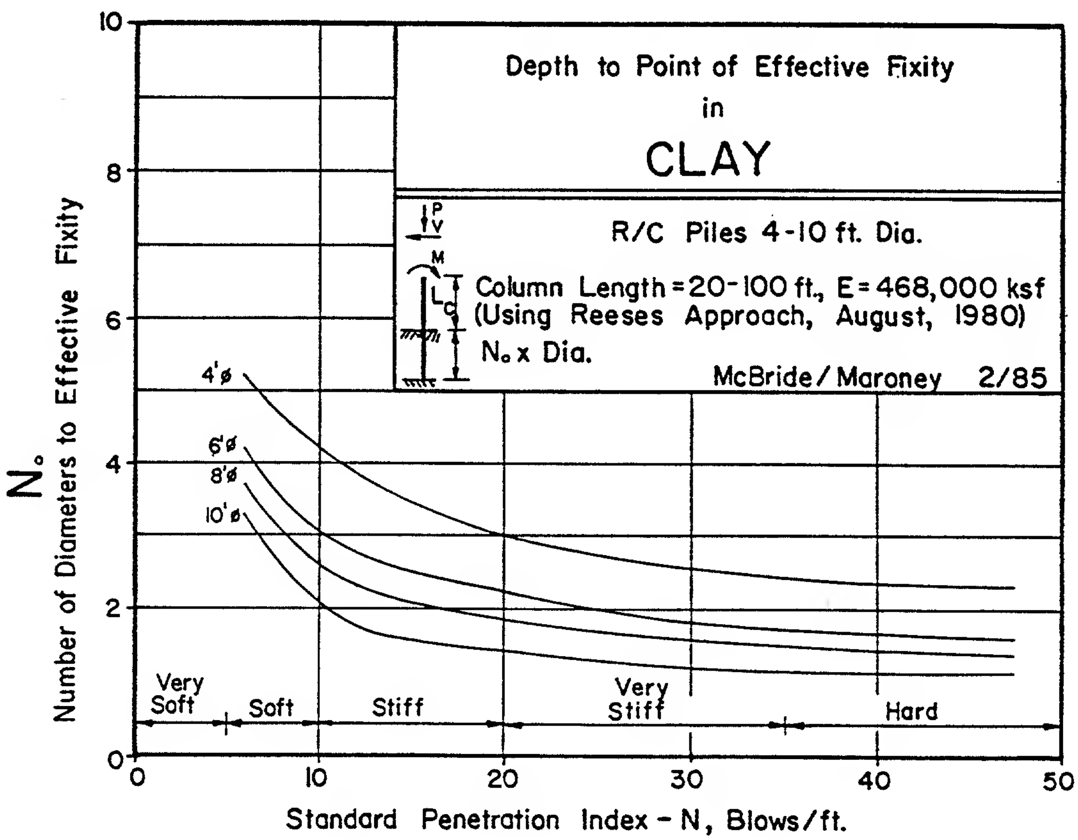
f_{max} = maximum unit friction, and

z_c = the critical movement of the pile segment at which f_{max} is fully mobilized. A z_c value of 0.2 in (0.5 cm) is recommended for all soil types.



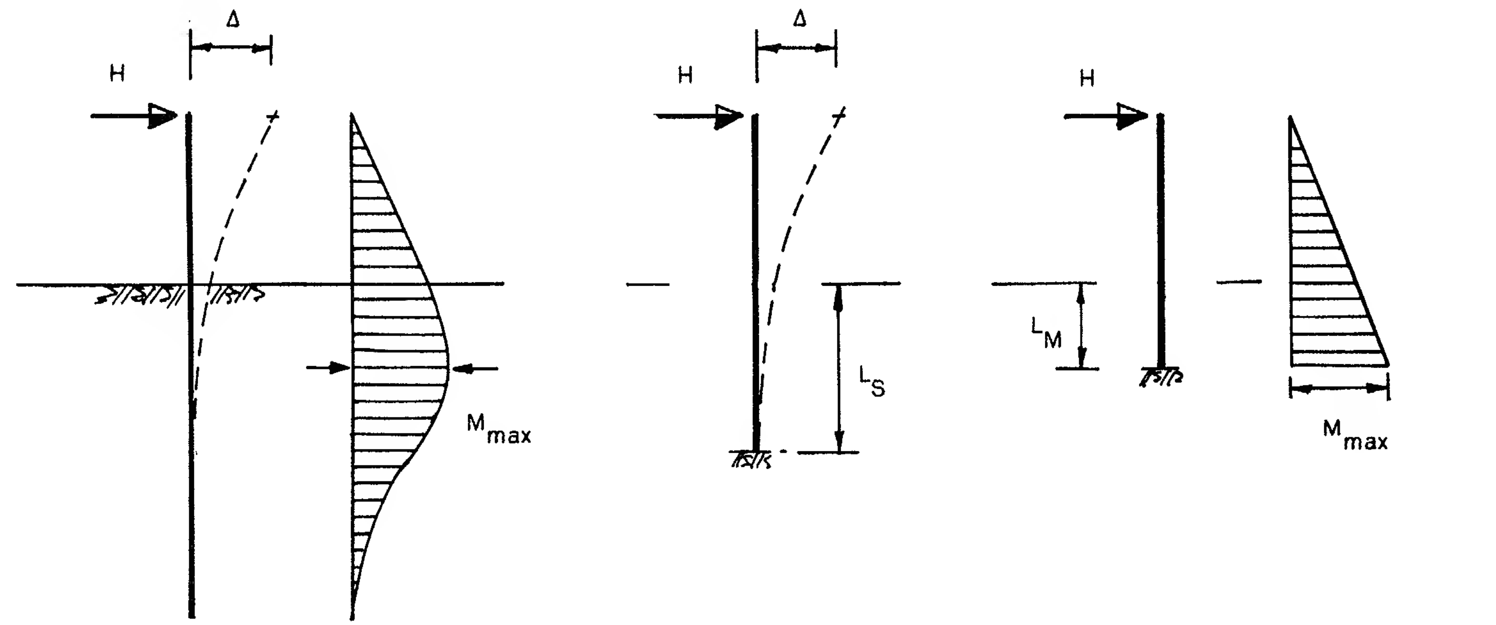
0-28	28-30	30-36	36-41	Approximate ø in degrees

Figure 68a: Depth to Point of Effective Fixity for Drilled Shafts in Sand (from Reference 35)



0-.5	.5-1	1-2	2-4	4-6 Shear Strength, ksf

Figure 68b: Depth to Point of Effective Fixity for Drilled Shafts in Clay (from Reference 35)



	L_S	L_M
Cohesive Soil Constant K_h	$1.4\sqrt[4]{\frac{EI}{K_h}}$	$0.44\sqrt[4]{\frac{EI}{K_h}}$
Cohesionless Soil Constant n_h	$1.8\sqrt[5]{\frac{EI}{n_h}}$	$0.78\sqrt[5]{\frac{EI}{n_h}}$

Figure 69: Equivalent Cantilever Method using Relative Stiffness Factors (after References 29 and 36)

End Bearing:

$$q = \left(\frac{z}{z_c}\right)^{1/3} q_{max} \tag{30}$$

where

q_{max} = maximum tip resistance

q = tip resistance mobilized at any value of $z \leqslant z_c$, and

z_c = critical displacement corresponding to q_{max}. A z_c value of 0.05 of the pile diameter is recommended.

Computer programs have been written to develop axial stiffness constants based on these and other more refined equations.

7.2.4C Drilled Shafts (Piers)

A drilled shaft or pier is frequently used to support a single column bent and may be considered to act as a single vertical pile. However, this shaft is usually of larger diameter than the column and is, therefore, somewhat larger (in diameter) than a conventional pile.

Nevertheless, analysis of behavior and modelling can be treated in a similar manner to that discussed in section 7.2.4B for piles, provided due consideration is given to the differences in size, member stiffness and installation methods. These factors are discussed in references 6 and 35. Figures 68a and 68b give effective depths to fixity for large diameter reinforced concrete piles (4 to 10 ft diameter) and are therefore suitable for use in the design of drilled shafts (reference 35).

7.2.4D Abutments

For bridges that transfer loads through the abutments, careful attention must be given to abutment modelling. There are numerous case histories of bridges damaged or rendered unusable by excessive abutment displacements or abutment failures.

As noted in chapter 5, there are two principal types of abutments: the monolithic or end diaphragm abutment (figure 53a) and the seat type abutment (figure 53b). Equivalent spring models for both types are shown in figure 71, but these may be changed or modified to suit particular conditions. The intent is to represent the force-displacement relationship at the abutments but this is a highly complex non-linear problem, affected by both the soil properties and the design of the abutment itself. Both the configuration of the springs and their properties should reflect these conditions. Calculation of the equivalent spring constants is, therefore, a complex process but in the absence of more accurate information, the following iterative technique from the AASHTO Guide Specification may be used to determine equivalent properties. The procedure is outlined in a flow chart in figure 72 and is described in the following steps. A numerical example is given in appendix B.

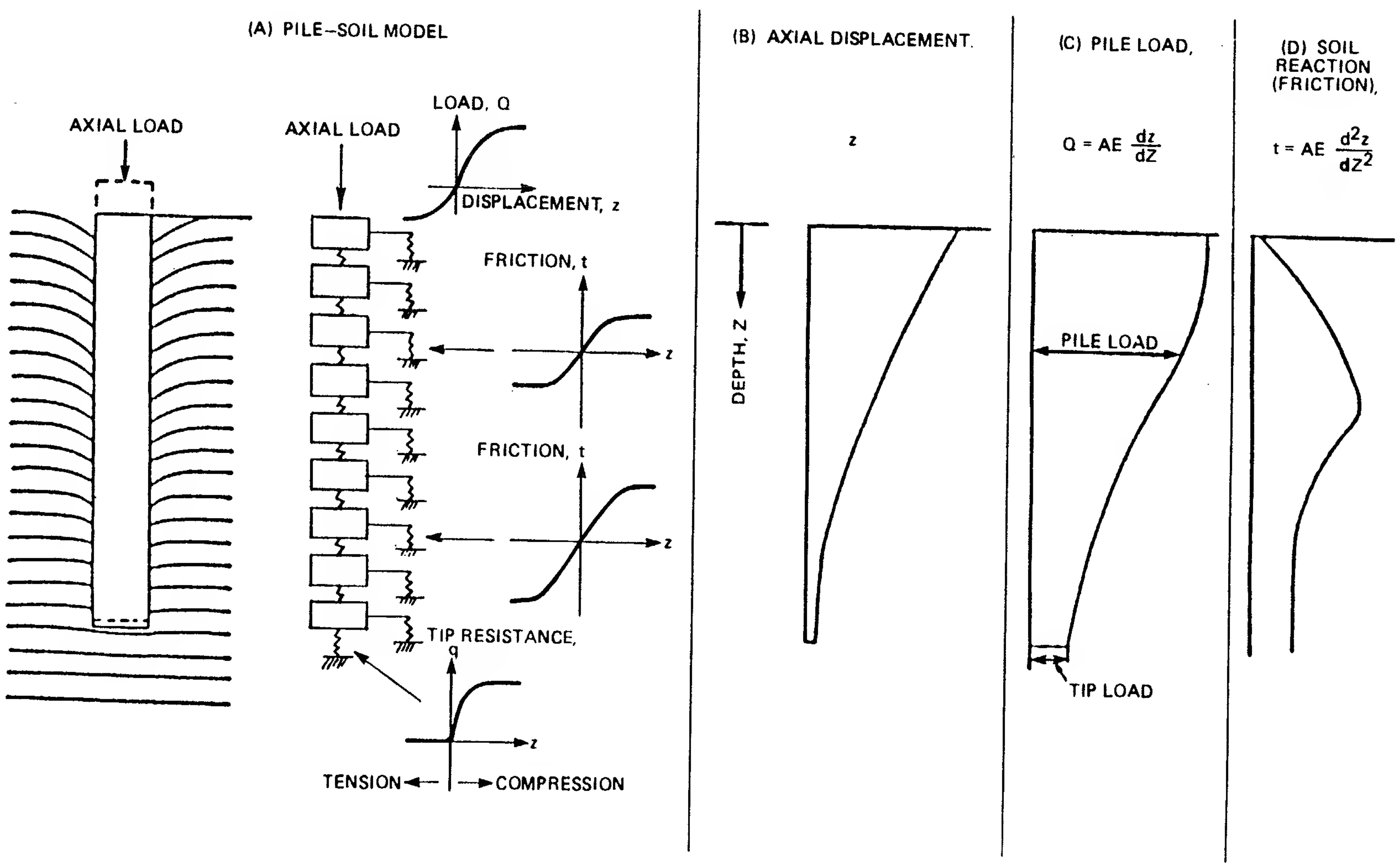
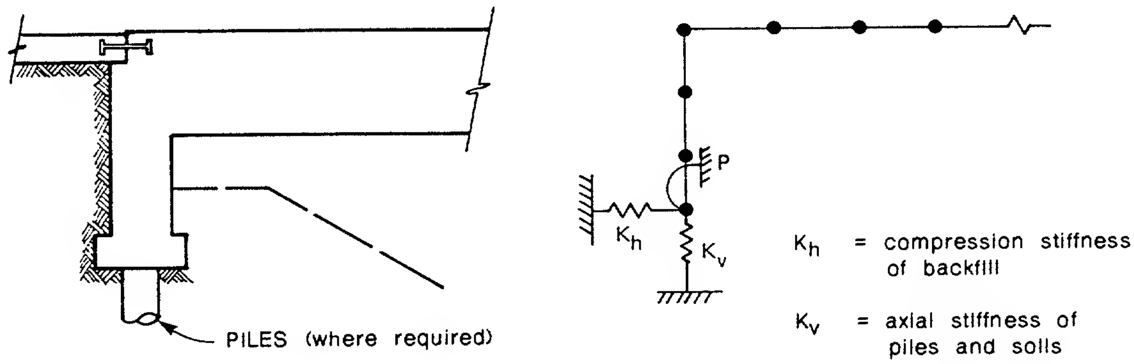
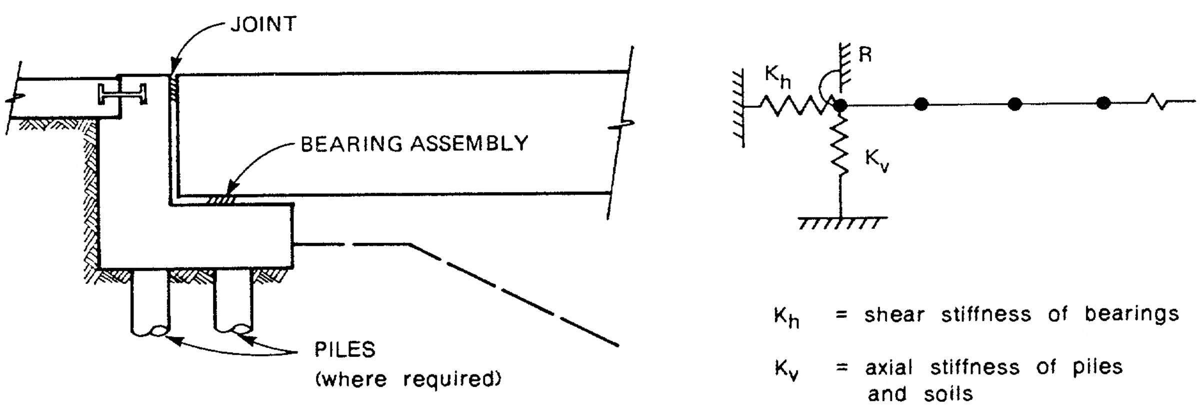


Figure 70: Representation of Axially Loaded Pile Behavior and Soil Reaction (from Reference 6)



(a) Monolithic Abutment



(b) Seat-type Abutment

Figure 71: Abutment Models (after Reference 6)

(1) Assume an initial abutment design and stiffness.

(2) Analyze the bridge and determine the forces at the abutment. Perform one of the following steps:

    (a) If the force levels exceed the acceptable capacity of the abutment fill and/or piles, reduce the stiffness of the abutments until the analysis indicates force levels below the acceptable capacity.

    (b) If the force levels are below the acceptable capacity of the abutment, proceed to step 3.

(3) Examine the calculated displacements at the abutment and perform one of the following steps:

    (a) If the displacements exceed acceptable levels, the assumed abutment design is inadequate. Redesign the abutment and return to step 1.

    (b) If displacements are acceptable, the last assumed abutment stiffness is consistent with the assumed abutment design. Use the results from this analysis.

Abutment design is discussed in more detail in references 4, 6 and 38.

7.3 COMBINATION OF RESULTS FROM ORTHOGONAL ANALYSES

A combination of forces from orthogonal seismic analyses is used to account for the directional uncertainty of future earthquake motions, and the simultaneous occurrence of earthquake motions in two perpendicular horizontal directions. The elastic seismic forces calculated from analyses considering separate input in each of two orthogonal directions should be combined to form two load cases as follows:

(1) **Load Case 1**: 100 percent of the absolute values of force and moment from the analysis in one of the perpendicular (longitudinal) directions are added to 30 percent of the absolute value of the corresponding forces and moments from the analysis in the second (transverse) direction. Absolute values are used because the direction of seismic response can be positive or negative.

(2) **Load Case 2**: 100 percent of the absolute value of force and moment from the analysis in the second perpendicular (transverse) direction are added to 30 percent of the absolute value of the corresponding forces and moments from the analysis in the first (longitudinal) direction.

7.4 SPECIAL REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY A

These requirements and those in sections 7.5 and 7.6 are taken from the AASHTO Guide Specifications [reference 4]. The basic distinction between these provisions and

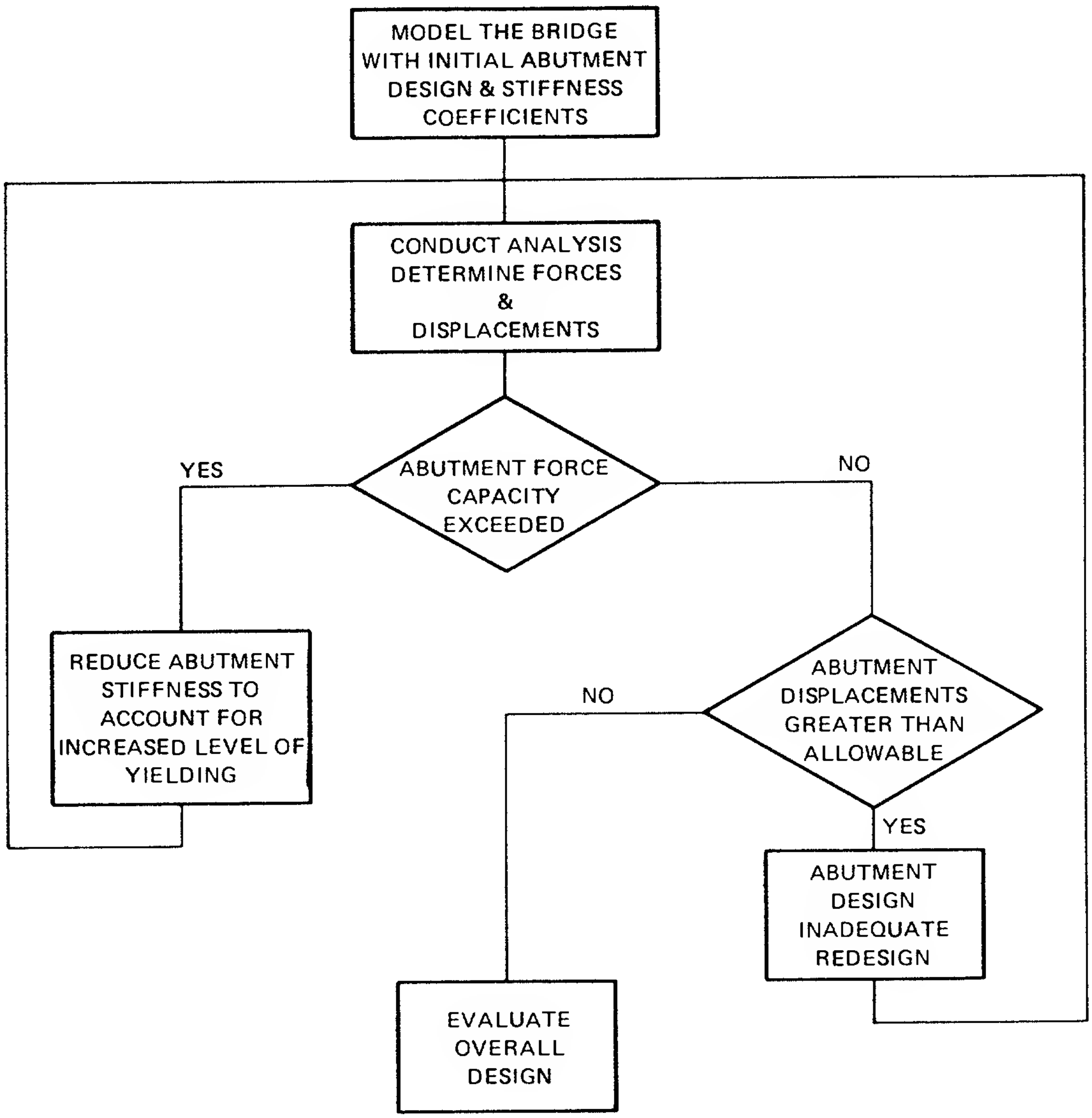


Figure 72: Iterative Procedure for the Determination of Abutment-Soil Interaction (from Reference 4)

those of the AASHTO Standard Specifications [reference 1] is that here the response is calculated from an elastic analysis of the structure using earthquake loads which have **not** been reduced to approximate the effects of ductility. Reductions are applied **separately** to individual member elastic forces, reflecting the differing abilities of various portions of the structure to undergo inelastic deformations. It also reflects the consequences to the integrity of the structure, should these deformations occur. Also, the approach adopted herein gives deflections of realistic magnitude, whereas the AASHTO Standard Specifications severely underestimate deflections due to seismic loads.

7.4.1 Design Forces

The connection of the superstructure to the substructure should be designed to resist a horizontal seismic force equal to 0.2 times the dead load reaction force in the restrained directions. Note that single span bridges do not need to satisfy this requirement, as discussed in section 7.7.

7.4.2 Design Displacements

Bearing seats supporting the expansion ends of girders, as shown in figure 73, should be designed to provide a minimum support length, N (inches or mm) measured normal to the face of the abutment or pier, as specified below.

$$N = 8 + 0.02\,L + 0.08\,H \qquad \text{(Inches)} \tag{31A}$$

or

$$N = 200 + 1.67\,L + 6.67\,H \qquad \text{(mm)} \tag{31B}$$

where

L = length, in feet for equation 31A or meters for equation 31B, of the bridge deck from the seat under consideration to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L should be the sum of L_1 and L_2, the distances on each side of the hinge to an adjacent expansion joint or end of deck. For single span bridges, L equals the length of the bridge deck. These lengths are also shown in figure 73.

For abutments

H = average height, in feet for equation 31A or meters for equation 31B, of those columns supporting the bridge deck from this abutment, to the next expansion joint. H = 0 for single span bridges.

For columns and/or piers

H = column or pier height in feet for equation 31A or meters for equation 31B.

For hinges within a span

H = average height of the adjacent two columns or piers in feet for equation 31A or meters for equation 31B.

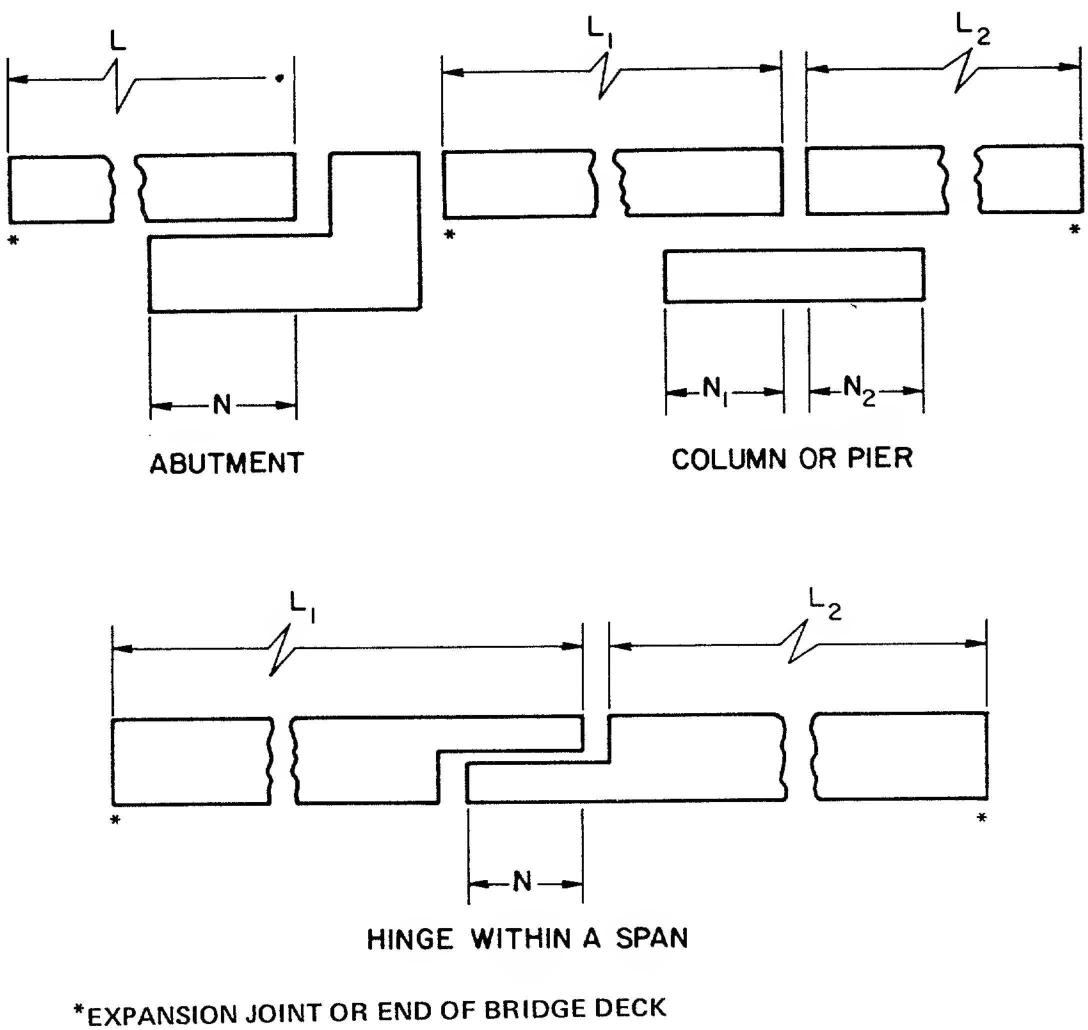


Figure 73: Dimensions for Minimum Support Length Requirements (from Reference 4)

7.5 SPECIAL REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY B

7.5.1 Design Forces for Structural Members and Connections

Seismic design forces given in this subsection apply to:

(a) The superstructure, its expansion joints and the connections between the superstructure and the supporting substructure.

(b) The supporting substructure down to the base of the columns and piers but not including the footing, pile cap or piles.

(c) Components connecting the superstructure to the abutment.

Seismic design forces for the above components should be determined by dividing the elastic seismic forces obtained from Load Case 1 and Load Case 2 of section 7.3 by the appropriate Response Modification Factor of section 6.4. The modified seismic forces resulting from the two load cases are then combined independently with forces from other loads as in the following group loading combination for the components. Note that the seismic forces are reversible (positive and negative).

$$\text{Group Load} = 1.0\ (D + B + SF + E + EQM) \tag{32}$$

where
- D = dead load
- B = bouyancy
- SF = stream-flow pressure
- E = earth pressure
- EQM = elastic seismic force for **either** Load Case 1 **or** Load Case 2 of section 7.3 modified by dividing by the appropriate R-Factor (section 6.4).

Each component of the structure is designed to withstand the forces resulting from each load combination according to the AASHTO Standard Specifications. Note that equation 32 should be used in lieu of the AASHTO Group VII loading combination and that the γ and β factors equal 1. For Service Load Design, a 50 percent increase is permitted in the allowable stresses for structural steel and a 33-1/3 percent increase for reinforced concrete.

7.5.2 Design Forces for Foundations

Seismic design forces for foundations, including footings, pile caps, and piles should be the elastic seismic forces obtained from Load Case 1 and Load Case 2 of section 7.3 divided by the Response Modification Factor (R) specified below. These modified seismic forces are then combined independently with forces from other loads as indicated in the following group loading combination to determine two alternate load combinations for the foundations.

$$\text{Group Load} = 1.0\ (D + B + SF + E + EQF) \tag{33}$$

where D, B, SF and E are as defined in section 7.5.1 and

EQF = the elastic seismic force for **either** Load Case 1 **or** Load Case 2 of section 7.3 divided by half the R-Factor for the substructure (column or pier) to which it is attached.

EXCEPTION:
For pile bents the R-Factor should not be divided by 2.

Each component of the foundation should be designed to resist the forces resulting from each load combination and the AASHTO Standard Specifications.

7.5.3 Design Forces for Abutments and Retaining Walls

The components (bearings, shear keys) connecting the superstructure to an abutment should be designed to resist the forces given in section 7.5.1.

7.5.4 Design Displacements

The seismic design displacements should be the maximum of those determined from the elastic analysis or those given in section 7.4.2.

7.6 SPECIAL REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D

Two sets of design forces are outlined in section 7.6.1 and 7.6.2 for bridges classified as Category C or D. The design forces for the various components are given in sections 7.6.3 through 7.6.7. The design displacements are provided in section 7.6.8.

7.6.1 Modified Design Forces

These should be determined as suggested in section 7.5.1 except that for columns a maximum and minimum axial force should be calculated for each load case by taking the seismic axial force as positive and negative.

7.6.2 Forces Resulting From Plastic Hinging

The forces resulting from plastic hinging at the top and/or bottom of the column should be calculated after the preliminary design of the columns is complete. The forces resulting from plastic hinging are recommended for determining design forces for most components as outlined in sections 7.6.3 through 7.6.6. Alternate conservative design forces are given if forces resulting from plastic hinging are not calculated. The procedures for calculating these forces for single column and pier supports and bents with two or more columns are given in the following subsections and are an implementation of the capacity design approach outlined in section 4.6.

Note that if the column moments do not reach their plastic values, the shear forces from plastic hinging will not govern. The governing design forces will then be those from the unreduced elastic spectrum or from other load groups.

A. Single Columns and Piers

The forces are calculated for the two principal axes of a column and in the weak

direction of a pier as follows:

Step 1. Determine the column overstrength plastic moment capacities. For reinforced concrete columns, use a flexural overstrength factor (Φ_O) of 1.3 and for structural steel columns use 1.25, nominal yield strength. (NOTE: This terminology is different to that used in section 4.8.2(A) of the AASHTO Guide Specification. This is done to minimize possible confusion with the traditional use of strength reduction factors and to be consistent with section 4.6 of this manual. However, the intent of both approaches is the same.) For both materials, use the maximum elastic column axial load from section 7.3 added to the column dead load.

Step 2. Using the column overstrength plastic moments, calculate the corresponding column shear force. For flared columns, this calculation should be performed using the overstrength plastic moments at both the top and bottom of the flare with the appropriate column height. If the foundation of a column is significantly below ground level, consideration should be given to the possibility of the plastic hinge forming above the foundation. If this can occur, the column length between plastic hinges should be used to calculate the column shear force. Recommended column lengths as used by Caltrans for a variety of pier configurations, are shown in figure 74.

The forces corresponding to a single column hinging are:

1. Axial Forces – unreduced maximum and minimum seismic axial load of section 7.3 plus the dead load.

2. Moments – those calculated in Step 1.

3. Shear Force – that calculated in Step 2.

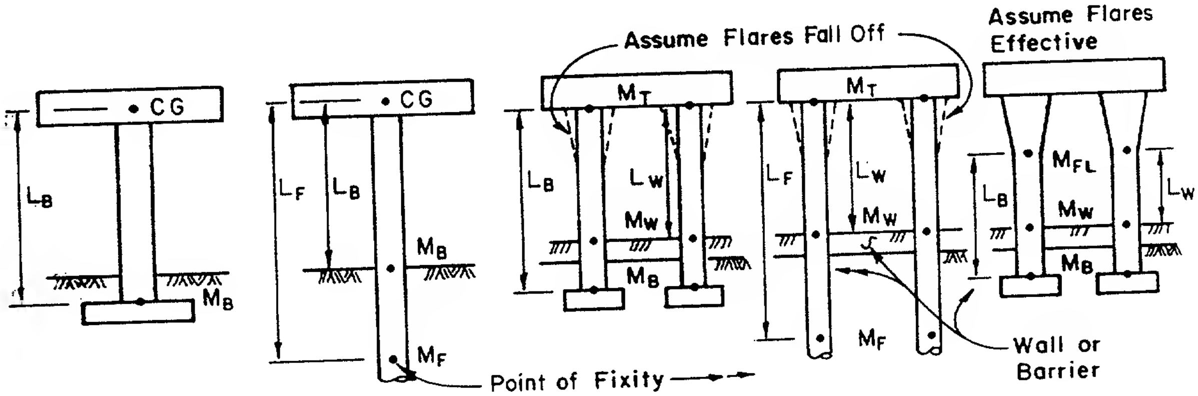
B. Bents with Two or More Columns

The forces for bents with two or more columns should be calculated both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent the forces are calculated as for single columns in section 7.6.2(A). In the plane of the bent the forces may be calculated as follows:

Step 1. Determine the column overstrength plastic moment capacities. For reinforced concrete, use a flexural overstrength factor (Φ_O) of 1.3 and for structural steel use 1.25. (NOTE: As noted in (A) above, there is a change in terminology here from that used in the AASHTO Guide Specifications. This is done to be consistent with section 4.6.) For both materials, use the axial load corresponding to the dead load.

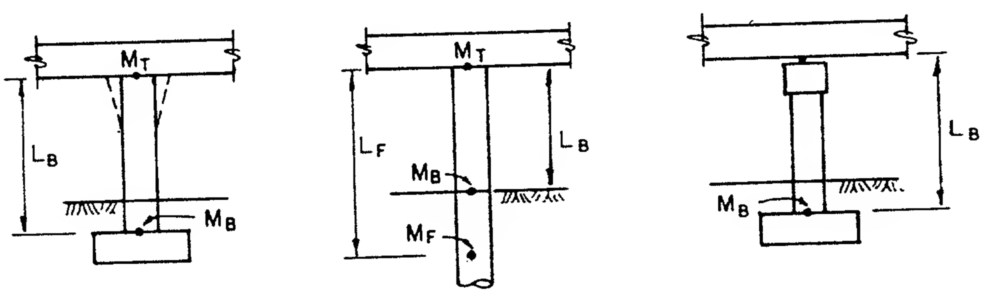
Step 2. Using the column overstrength plastic moments calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the bent. Note that, if a partial-height wall exists between the columns, the effective column height is taken from the top of the wall. For flared columns and foundations below ground level, see section 7.6.2(A) Step 2 and figure 74. For pile bents, the length of pile above the mud line should be used to calculate the shear force.

Step 3. Apply the bent shear force to the top of the bent (center of mass of the superstructure above the bent) and determine the axial forces in the columns due to



$$V = \frac{M_B}{L_B} \qquad V = \frac{M_B}{L_B} \text{ or } \frac{M_F}{L_F} \qquad V = \frac{M_T + M_W}{L_W} \text{ or } = \frac{M_T + M_B}{L_B} \qquad V = \frac{M_T + M_W}{L_W} \text{ or } = \frac{M_T + M_F}{L_F} \qquad V = \frac{M_{FL} + M_W}{L_W} \text{ or } = \frac{M_{FL} + M_B}{L_B}$$

(a) TRANSVERSE



$$V = \frac{M_T + M_B}{L_B} \qquad V = \frac{M_T + M_B}{L_B} \text{ or } \frac{M_T + M_F}{L_F} \qquad V = \frac{M_B}{L_B}$$

(b) LONGITUDINAL

Figure 74: Calculation of Shear Force for Different Plastic Hinge Locations (from Reference 38)

overturning when the column overstrength plastic moments are developed.

Step 4. Using these column axial forces combined with the dead load axial forces, determine revised column overstrength plastic moments. With the revised overstrength plastic moments, calculate the column shear forces and the maximum shear force for the bent. If the maximum shear force for the bent is not within 10 percent of the value previously determined, use this maximum bent shear force and return to Step 3.

The forces in the individual columns in the plane of a bent corresponding to column hinging, are:

1. Axial Forces – the maximum and minimum axial load is the dead load plus, or minus, the axial load determined from the final iteration of Step 3.

2. Moments – the column overstrength plastic moments corresponding to the maximum compressive axial load specified in (1) with an overstrength factor of 1.3 for reinforced concrete and 1.25 for structural steel.

3. Shear Force – the shear force corresponding to the column overstrength moments in (2), noting the provisions in Step 2 above.

7.6.3 Design Forces for Column Bents and Pile Bents

Design forces for columns and pile bents should be the following:

(a) Axial Forces – the minimum and maximum design force should be either the elastic design values determined in section 7.3 added to the dead load, or the values corresponding to plastic hinging of the column as determined in section 7.6.2. Generally, the values corresponding to column hinging will be smaller and it is then recommended that these smaller values be used.

(b) Moments – the modified design moments determined in section 7.6.1.

(c) Shear Force – either the elastic design value determined from section 7.6.1 using an R-Factor of 1 for the column or the value corresponding to plastic hinging of the column as determined in section 7.6.2. Generally, the value corresponding to column hinging will be significantly smaller and it is then recommended that this smaller value be used.

7.6.4 Design Forces for Wall Piers

The design forces should be those calculated in section 7.6.1 except if the pier is designed as a column in its weak direction. In this case, the design forces in the weak direction are those in section 7.6.3 and all the design requirements for columns of chapter 8 of the AASHTO Guide Specifications are applicable. (NOTE: When the forces due to plastic hinging are used in the weak direction, the combination of forces specified in section 7.3 is not applicable.)

7.6.5 Design Forces for Connections

The design forces should be those determined in section 7.6.1 except that for superstructure connections to columns and column connections to cap beams or footings, the alternate forces specified in (C) below are recommended. Additional design forces at connections are as follows:

A. Longitudinal Linking Forces

A positive horizontal linkage is recommended between adjacent sections of the superstructure at supports and expansion joints within a span. The linkage is designed for a minimum force of the Acceleration Coefficient times the weight of the lighter of the two adjoining spans or parts of the structure. If the linkage is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motions, sufficient slack should be allowed in the linkage so that the linkage force does not start to act until the design displacement is exceeded. Where a linkage is to be provided at columns or piers, the linkage of each span may be attached to the column or pier rather than between adjacent spans. Positive linkage may be provided by ties, cables, dampers or equivalent mechanisms. Friction should not be considered a positive linkage.

B. Hold-Down Devices

Hold-down devices are recommended at all supports or hinges in continuous structures, where the vertical seismic force due to the longitudinal horizontal seismic load opposes and exceeds 50 percent but is less than 100 percent of the dead-load reaction. In this case the minimum net upward force for the hold-down device should be at least 10 percent of the dead load downward force that would be exerted if the span were simply supported.

If the vertical seismic force (Q) due to the longitudinal horizontal seismic load opposes and exceeds 100 percent of the dead load reaction (DR), the net upwards force for the hold-down device should be at least 1.20(Q-DR) and not be less than that given in the previous paragraph.

C. Column and Pier Connection Design Forces

The recommended connection design forces between the superstructure and columns, columns and cap beams, and columns and spread footings or pile caps are the forces developed at the top and bottom of the columns due to column hinging as determined in section 7.6.2. The smaller of these or the values specified in section 7.6.1 may be used. Note that these forces should be calculated after the column design is complete and the overstrength moment capacities have been obtained.

7.6.6 Design Forces for Foundations

The design forces for foundations including footings, pile caps and piles may be either those forces determined in section 7.6.1 or the forces at the bottom of the columns corresponding to column plastic hinging as determined in section 7.6.2. Generally, the values corresponding to column hinging will be significantly smaller and then these smaller values are recommended for design.

When the columns of a bent have a common footing, the final force distribution at the base of the columns in Step 4 of section 7.6.2(B) may be used for the design of the footing in the plane of the bent. This force distribution produces lower shear forces and moments on the footing because one exterior column may be in tension and the other in compression due to the seismic overturning moment. This effectively increases the ultimate moments and shear forces on one column and reduces them on the other.

7.6.7 Design Forces for Abutments and Retaining Walls

The components (bearings and shear keys) connecting the superstructure to an abutment should be designed to resist the forces specified in section 7.6.1.

7.6.8 Design Displacements

The seismic design displacements should be the maximum of those determined from the elastic analysis or those given in section 7.4.2 except that equations 31A and 31B are replaced by:

$$N = 12 + 0.03L + 0.12H \quad \text{(inches)} \tag{34A}$$

or

$$N = 300 + 2.5L + 10H \quad \text{(mm)} \tag{34B}$$

where N, L and H are defined in section 7.4.2.

Positive horizontal linkage, as recommended in section 7.6.5, should be provided in all superstructure gaps or expansion joints within a span.

Relative displacements between different segments of the bridge should be carefully considered in the evaluation of the results determined from the elastic analysis. Relative displacements arise from effects that are not easily included in the analysis procedure but should be considered in determining the design displacements. They include the following:

(a) Displacements due to rotation of bridge decks on skew supports (torsional displacements).

(b) Rotation and/or lateral displacements of the foundations.

(c) Out-of-phase displacements of different segments of the bridge. This is especially important in determining seat widths at expansion joints.

(d) Out-of-phase rotation of abutments and columns induced by travelling seismic waves.

The stability of pile bents and single pile shafts should be checked if plastic hinging is expected.

7.7 EXEMPTIONS FOR SINGLE SPAN BRIDGES

A detailed seismic analysis is not necessary for single span bridges regardless of its Seismic Performance Category. However, the connections between the bridge span and the abutments should be designed both longitudinally and transversely to resist the gravity reaction force at the abutments multiplied by the acceleration coefficient of the site. Further, the minimum support lengths should be as given in either section 7.4.2 for a bridge in Seismic Performance Category A or B, or section 7.6.8 for a bridge in Seismic Performance Category C or D.

7.8 COMPUTER PROGRAMS

When selecting a computer program for seismic analysis, it is important to remember that seismic loads are lateral loads which are primarily resisted by the substructure. Consequently, they have little influence on superstructure stresses. It is therefore important that the program be able to model the substructure components and foundations with some degree of sophistication and it is less important to model the superstructure beyond a basic level. Most of the computer programs developed so far for bridge analysis are typically oriented towards vertical or gravity load analyses and for this purpose some very sophisticated codes have been developed (finite element models and grillage models are examples of these). However, these refinements are not required for seismic analysis and simple space frame models can be quite adequate. Of course the replacement of the superstructure by a single beam requires some careful assessment of equivalent beam properties but the error introduced by this approximation on the seismic response of the bridge is not significant.

General purpose space frame programs are therefore satisfactory for the seismic analysis of bridge structures. Software which has been used for this purpose includes SAP IV, STRUDL/DYNAL, and EAC/EASE2. However, since these are all general purpose programs, they are not specifically oriented towards the bridge designer and data input can be tedious and interpretation of the output frustrating. The terminology used to describe both the structure and the analysis is not familiar to bridge engineers and the software may appear, at first sight, to be unsuitable for seismic bridge analysis. In short, these programs are not user-friendly. The possible exception here is STRUDL, but even so, it is not bridge specific and some interfacing and pre-processing of the input is required before a bridge engineer can take his data from the drawing board and input it to the computer.

To meet an obvious need, and also to include some analytical procedures unique to bridge seismic analysis, SEISAB has been developed by the Engineering Computer Corporation. This software combines the dialogue of STRUDL with the numerical efficiency of SAP IV and produces a code specifically developed for bridge engineers, which is user-friendly and readily available.

Short notes on each of the above mentioned computer programs follow.

7.8.1 SAP IV [reference 39]

SAP is a structural analysis program for the static and dynamic analysis of linear systems.

SAP IV is the version of SAP originally developed at the University of California at Berkeley. This version does not have many of the user enhancements available in the later versions (SAP V and SAP VI), but it does have the capability of solving most of the linear dynamic analysis problems encountered by the bridge designer. It also is a relatively simple program from a computer programmer's point of view and may be easily modified to suit the needs of a particular group of users.

Structural systems that can be analyzed may be composed of combinations of a number of different structural elements. As a minimum, the program library contains the following element types:

(a) three-dimensional truss element.
(b) three-dimensional beam element.
(c) plane stress and plane strain element.
(d) two-dimensional axisymmetric solid.
(e) three-dimensional solid.
(f) thick shell element.
(g) thin plate or thin shell element.
(h) boundary element.
(i) pipe element (tangent and bend).

These structural elements can be used in a static or dynamic analysis. The capacity of the program depends mainly on the total number of nodal points in the system, the number of eigenvalues (modal frequencies) needed in the dynamic analysis and the computer used. There is practically no restriction on the number of elements used, the number of load cases or the order and bandwidth of the stiffness matrix. Each nodal point in the system can have from zero to six displacement degrees of freedom. The element stiffness and mass matrices are assembled in condensed form. The program is therefore equally efficient in the analysis of one-, two-, or three-dimensional systems.

7.8.2 STRUDL/DYNAL [reference 40]

ICES STRUDL, (STRUctural Design Language), is a large general purpose structural analysis and design computer system. The code is designed to allow the user to define the problem in terms that are familiar to the structural engineer. It is important, however, that the user understand the command structure and the way the computer will interpret the commands to solve a typical problem.

Seismic analysis is carried out using the dynamic analysis options of STRUDL/DYNAL. Quite sophisticated problems in structural dynamics can be solved using this program, but it is not as efficient to use as SAP IV. Several preprocessors for STRUDL have been developed to facilitate its use in specialized circumstances. Examples of these for bridge analysis are STRUBAG (developed by Caltrans) and BRIDGEN (developed by McAuto).

7.8.3 EAC/EASE2

EAC/EASE2 is a static or dynamic, finite element, linear analysis program. Emphasis is on the ease of input, utility of output and cost effectiveness. The program is particularly well-suited for the efficient analysis of very large structural models. Operational on CDC CYBER 70, 170 and 6000-Series computer systems, the program

is available in batch mode through Control Data's CYBERNET Services, and McDonnell Douglas Automation Company.

The analysis options available in EASE2 are as follows:

Static loading conditions include temperature, thermally induced bending in beams and shells, normal pressures and edge tractions on membranes and shells, distributed beam and pipe loads, and face pressures on solid elements.

Dynamic loading conditions include external time dependent loads or base (ground) acceleration time histories. Response spectra and direct integration methods are available.

7.8.4 SEISAB [reference 41]

SEISAB (SEISmic Analysis of Bridges) is a computer program specifically developed for the seismic analysis of bridges. The overall objectives in developing SEISAB were to provide the practicing bridge engineer with a usable design tool and a vehicle for implementing the latest seismic design methodologies into the bridge engineering profession.

The initial program release, SEISAB-I, offers both the single mode and multi-mode analysis options as described in the AASHTO Guide Specifications. It can also be used to perform the uniform load method which is inherent in the AASHTO Standard Specification.

SEISAB-I can analyze simply supported or continuous girder-type bridges with no practical limitation on the number of spans or the number of columns at a bent. In addition, earthquake restrainer units may be placed between adjacent structural segments. Horizontal alignments composed of a combination of tangent and curved segments are described using alignment data taken directly from roadway plans. SEISAB has generating capabilities that will, with a minimum of input data, provide a consistent model appropriate to the analysis method selected. Seismic loadings in the form of response spectra are stored in the system and may be directly referenced by the user.

The central theme underlying the development of SEISAB was to provide the bridge designer with an effective means of user-program communication using a problem-oriented language. This free format input language consists of simple, easy to remember commands natural to the bridge engineer. User input data is checked for syntax and consistency prior to conducting an analysis. In addition, numerous default values are assumed for data not entered by the user.

The bridge examples given in chapter 10 illustrate the usefulness and versatility of SEISAB. Sample input files show the user-friendly nature of this program.

It is intended that SEISAB-II will include linear and nonlinear transient analysis capabilities for the designer or researcher interested in conducting more detailed studies for assessment of new seismic design strategies.

CHAPTER 8 DESIGN EXAMPLES

Chapter 6 discussed the design loads on bridges in different seismic regions and chapter 7 the methods of determining design forces and displacements from these loads. In this chapter the application of these procedures is illustrated by performing design calculations for example bridges.

The basic bridge configuration used to illustrate the procedures is shown in figure 75. It is a three-span continuous box girder bridge with 2 seat-type abutments and 2 piers each of which are 3 column bents. The box girder is constructed monolithically with the bents.

Within the basic layout shown in figure 75 a number of variations and their effect on the design process are considered in this chapter:

- Location of the bridge in different seismic zones - Examples 1 and 2.
- Changes in superstructure weight as would occur, for example, with girder or truss superstructure construction - Examples 3 and 4.
- Effect of using bearings between the top of the pier bents and the superstructure instead of monolithic construction - Examples 5 to 9.

The seismic zone in which the bridge is located has the greatest impact on the design procedures and forces and the majority of this chapter follows the design process for the basic bridge configuration in two different seismic zones. Example 1 is located within the 0.4 contour and classified as Seismic Performance Category D. Example 2 is located within the 0.1 contour and classified as Seismic Performance Category B. The two examples are presented together in section 8.1 in order to illustrate both the similarities and the differences in the calculations required.

The design examples presented in section 8.1 consist of a box girder deck that is cast monolithic with the columns as shown in figure 75. As discussed in chapter 5, there are many different types of deck and bearing configurations that could have been considered. Variations in bearing restraint for 3-span bridges are shown in figure 51c. Different superstructure types include steel or concrete girders and steel trusses. The intent of sections 8.2 to 8.4 is to evaluate the impact of some of these variations on the seismic response of the bridge used for the first two examples.

The configuration and member sizes for the bridge used in Examples 1 and 2 is identical to that given in appendix A of the AASHTO Guide Specifications [reference 4]. However these Specifications have an error in the calculation of the column capacity and so the column reinforcing required in Example 1 is higher than in the Guide

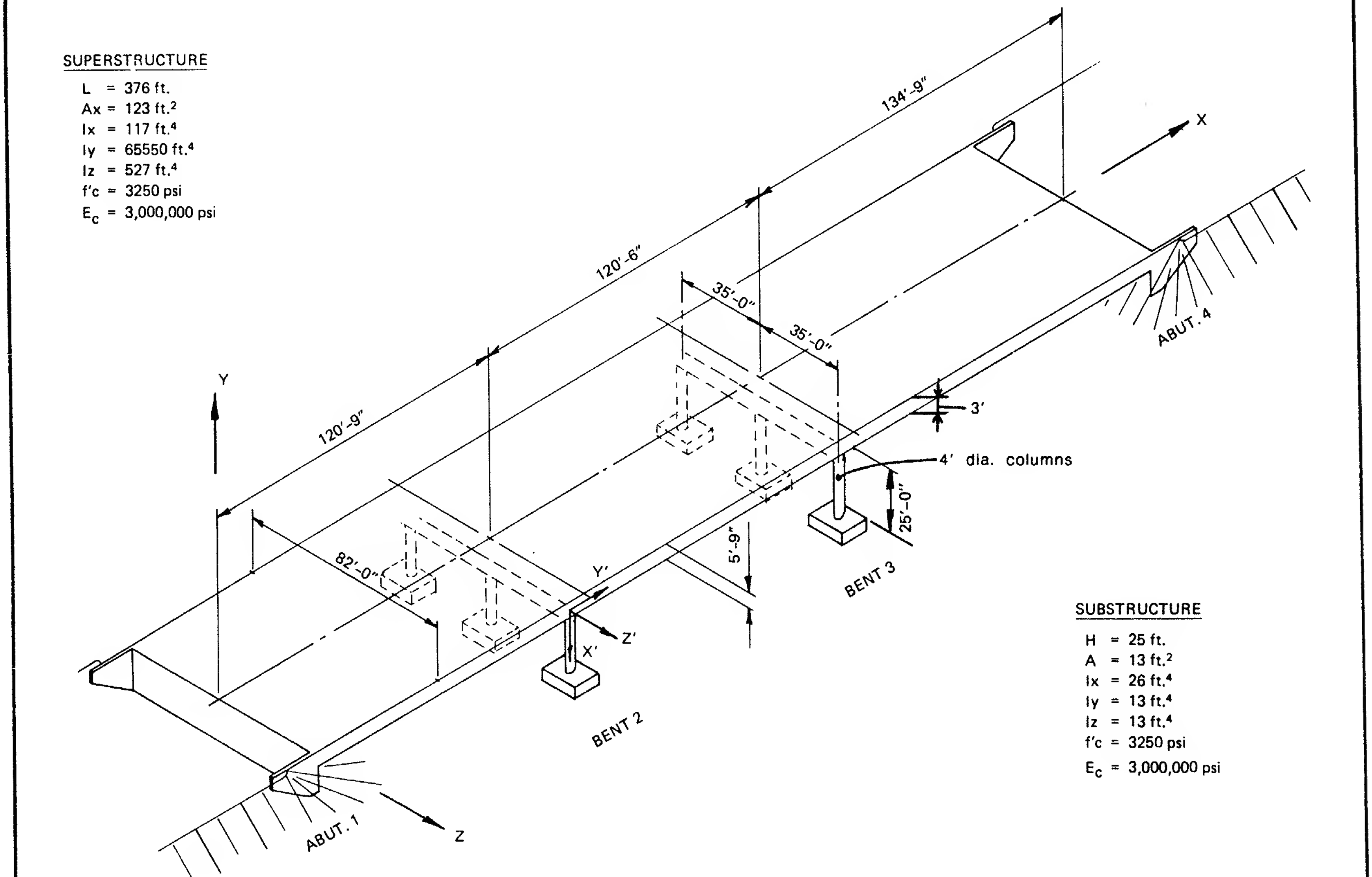


Figure 75: Dimensions of Example Bridge (from Reference 4)

Specifications. This is discussed further in section 8.1.10C.

The design calculations in these examples are for seismic loads plus dead loads only. In many instances the design of bridges will be governed by other group loads which do not include seismic loads.

8.1 EFFECT OF SEISMIC ZONE : EXAMPLE 1 and 2

For these examples the bridge shown in figure 75 is assumed to be located firstly in a region of high seismicity, e.g. California (**Example 1**) and secondly in a region of low to moderate seismicity, e.g. New York State (**Example 2**).

The AASHTO Guidelines [reference 4] are applicable to a box girder bridge with the alignment, dimensions and member properties shown in figure 75 and so form the basis for the design calculations performed in this chapter.

8.1.1 Acceleration Coefficient

Example 1 is located within the 0.4 contour (figure 55) and has an Acceleration Coefficient (A) equal to 0.40.

Example 2 is located within the 0.1 contour (figure 55) and has an Acceleration Coefficient (A) equal to 0.10.

8.1.2 Importance Classification

Example 1 is assumed to be essential in terms of Social/Survival and Security/Defense requirements and is therefore assigned an Importance Classsification (IC) of I (section 6.6).

Example 2 is assumed not to fall into any of the "essential" categories listed in section 6.6 and therefore has an Importance Classification of II.

8.1.3 Seismic Performance Category

Example 1, with $A > 0.29$ and an IC of I, falls into the Seismic Performance Category (SPC) D as shown in table 6 (section 6.5).

In **Example 2** for $0.09 < A \leq 0.19$ the Seismic Performance Category is B for both Importance Classifications.

8.1.4 Site Effects

Soil Profile II is assumed for the bridge site for **Example 1**, providing a Site Coefficient (S) of 1.2 from table 4 (section 6.2).

Soil Profile II is also assumed for **Example 2**, and therefore $S = 1.2$.

Note that this Soil Profile is also used if information is not available on the soil properties and profile.

8.1.5 Response Modification Factor

Substructure

The multiple column bent has a Response Modification Factor (R) of 5 for both orthogonal axes of the columns, as shown in table 5 (section 6.4).

Connections

Table 5 (section 6.4) provides an R-Factor for the superstructure to abutment connection of 0.8 and an R-Factor of 1.0 for the connection of the column to bent cap or superstructure and for the column to foundation.

Example 1 is classified as SPC D and the recommended design forces for column connections are those corresponding to the maximum force capable of being developed by column hinging as described in section 7.6.3. Therefore, the R-Factor for the column connections is not used since the forces resulting from column hinging are lower.

Example 2 is SPC B and so the R-Factors are used for all connections, using 0.8 for superstructure to abutment and 1.0 for column to bent cap or superstructure and for column to foundation.

Foundations

Example 1 is SPC D and so the foundation design forces are the lesser of those that result from plastic hinging of the columns or the elastic seismic forces.

Example 2 is SPC B and the elastic design forces are divided by an R-Factor of one-half that used for the substructure, i.e. R = 2.5.

8.1.6 Analysis Procedure

The bridge geometry and related stiffness variation falls within the range defined for a "regular bridge". As shown in table 7 (section 7.1.1), for a regular bridge with 2 or more spans Procedure 1 (Single Mode Spectral Analysis) is recommended as the minimum required analysis method for all Seismic Performance Categories (section 7.1.2). Thus, the method of analysis is the same for both examples. If the bridge had been irregular, the methods of the analysis would have differed. This same bridge is analyzed in Chapter 10 using Multimode Spectral Analysis (Procedure 2) techniques and a comparison of the results obtained by the two methods is presented in section 10.1.5.

8.1.7 Determination of Elastic Forces and Displacements

For both examples, earthquake motions are considered to act along the longitudinal and transverse axes of the bridge. These are the global X and Z axes respectively, shown in figure 75. The local Y' and Z' axes of the columns are not necessarily required to coincide with the longitudinal and transverse axes of the bridge. However, for a straight bridge without skew columns, piers or abutments, it is recommended, for simplicity of calculation, that the local Y' axis of the column or pier coincide with the longitudinal axis of the bridge as shown in this example.

8.1.8 Single Mode Spectral Analysis Method - Procedure 1

The analysis method (section 7.1.2) for both examples leads to identical calculations from Step 1 to Step 3 for both the longitudinal and transverse directions. At Step 4 the elastic seismic design coefficient for both the transverse and longitudinal directions is found, and at Step 5 the member forces and displacements are calculated. These last two steps have different but directly proportional results for the two examples since both are a direct linear function of the Acceleration Coefficient. In the first example the Acceleration Coefficient is 0.4 and in the second it is 0.1.

8.1.8A Longitudinal Earthquake Loading

Step 1: Compute deflection $v_s(x)$ under a unit lateral load.

This step is identical for both examples.

The bridge may be idealized as shown in figure 76 if axial deformations are neglected and it is assumed that the deck behaves as a rigid member. Note that the bearing stiffness at the seat-type abutments has been neglected in this case and therefore there is no contribution from the abutments to the longitudinal stiffness of the bridge. This was done for simplicity and results in a conservative estimate of forces to the pier bents. Chapter 7 provides procedures to include the abutment stiffness.

Applying the assumed uniform longitudinal loading results in a constant displacement (i.e., $v_s(x)=v_s$) along the bridge. Assuming that the columns alone resist the longitudinal motion, the displacement may be calculated by assuming a column stiffness of $12\ EI/H^3$ in this direction. Using the column properties given in figure 75, the stiffness for Bents 2 and 3, denoted in figure 76 as k_1 and k_2 respectively, are calculated as:

$$k_1 = k_2 = 3\ \frac{(12EI)}{H^3} = 3 \times \frac{12 \times 432000}{25^3 \times 13} = 12940\ \text{kips/ft} \tag{35}$$

From which the displacement under the unit load is calculated as:

$$v_s = \frac{p_o L}{k_1 + k_2} = \frac{1 \times 376}{2 \times 12940} = 0.0145\ \text{ft} \tag{36}$$

Note that more accurate results may be obtained by either a computer analysis or from moment distribution calculations so as to include the flexibility of the superstructure, since the column tops are not completely fixed against end rotation. In this example the effect is slight and this refinement is not necessary. However, the axial loads listed in table 11 have been obtained from such a procedure.

Step 2: Compute integrals α, β, γ

This step is identical for both examples.

Assuming a weight density of 165 lb/ft^3, the dead weight per unit length for the superstructure is $w(x) = 0.165\ Ax = 0.165(123) = 20.3$ kips/ft. (Note that this weight density is higher than plain concrete to include the weight of the upper half of the

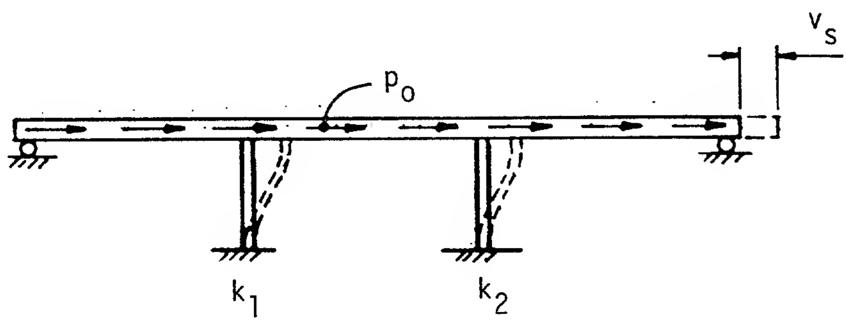


Figure 76: Structural Idealization and Application of Assumed Uniform Loading for Longitudinal Mode of Vibration (from Reference 4)

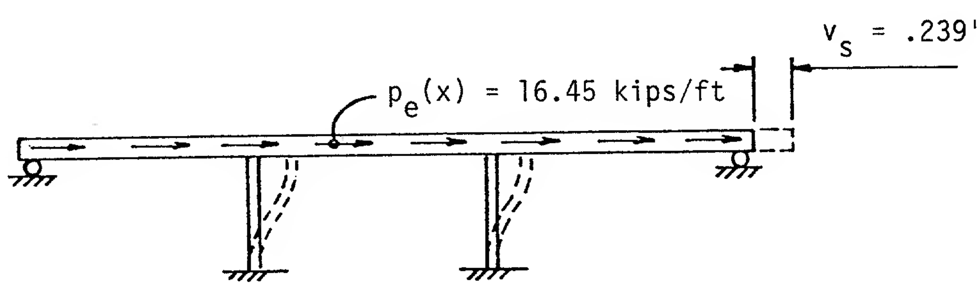


Figure 77: Displacements and Seismic Loading Intensity Longitudinal Loading (from Reference 4)

columns, the embedded column cap and intermediate diaphragms). The α, β, and γ factors are then calculated by evaluating the integrals in equations 23, 24 and 26 (section 7.1.2). For this case, both the dead weight per unit length $w(x)$, and the displacement, $v_s(x)$, are constant which simplifies the integration and yields:

$$\alpha = \int_{\text{Abut. 1}}^{\text{Abut. 4}} v_s(x)dx = v_s L = 0.0145 \times 376 = 5.46 \text{ ft}^2 \tag{37}$$

$$\beta = \int_{\text{Abut. 1}}^{\text{Abut. 4}} w(x)v_s(x)dx = wv_s L = 20.3 \times 0.0145 \times 376 = 110.9 \text{ kip-ft} \tag{38}$$

$$\gamma = \int_{\text{Abut. 1}}^{\text{Abut. 4}} w(x)v_s(x)^2dx = wv_s^2 L = 20.3 \times (.0145)^2 \times 376$$

$$= 1.61 \text{ kip-ft}^2 \tag{39}$$

Step 3: Compute the period, T

This step is identical for both examples.

The period, T, is calculated using equation 25.

$$T = 2\pi\sqrt{\frac{\gamma}{p_o g \alpha}} = 2\pi\sqrt{\frac{1.61}{[1.0 \times 32.2 \times 5.46]}} = 0.60 \text{ sec} \tag{40}$$

Step 4: Compute the seismic response coefficient, C_s

The elastic seismic response coefficient, C_s, is obtained from equation 22. Substituting for A, S and T yields:

Example 1: $A = 0.4$

$$C_s = \frac{1.2AS}{T^{2/3}}$$

$$= \frac{1.2 \times 0.4}{(0.60)^{2/3}} = 0.80 \tag{41}$$

Example 2 : $A = 0.10$

$$C_s = 0.20 \tag{42}$$

Since the seismic response coefficient does not exceed 2.5A for either example, use $C_s = 0.81$ for Example 1 and $C_s = 0.20$ for Example 2. The intensity of the seismic

loading expressed by equation 27 is therefore:

Example 1: $C_s = 0.81$

$$p_e(x) = \frac{\beta C_s w(x) v_s(x)}{\gamma}$$

$$= \frac{110.9 \times 0.81 \times 20.3 \times 0.0145}{1.61}$$

$$= 16.45 \text{ kips/ft} \tag{43}$$

Example 2: $C_s = 0.20$

$$p_e(x) = 4.11 \text{ kips/ft} \tag{44}$$

Step 5: Compute forces and displacements under static load

For **Example 1** the equivalent static loading is applied as shown in figure 77. The displacement and member forces for the longitudinal earthquake loading for this example (table 11) are obtained as follows:

Displacement

$$v_s = \frac{p_e(x) \cdot L}{k_1 + k_2} = \frac{16.45 \times 376}{2 \times 12960} = 0.239 \text{ ft.} \tag{45a}$$

$V_{Y'}$–Shear per Column

$$= \frac{16.45 \times 376}{6} = 1030 \text{ kips}$$

$M_{Z'Z'}$–Moment per Column

$$= 1030 \times 12.5 = 12{,}900 \text{ kip-ft} \tag{45b}$$

Note that for this bridge $V_{Z'}$ and $M_{Y'Y'}$ are zero for the longitudinal earthquake motion.

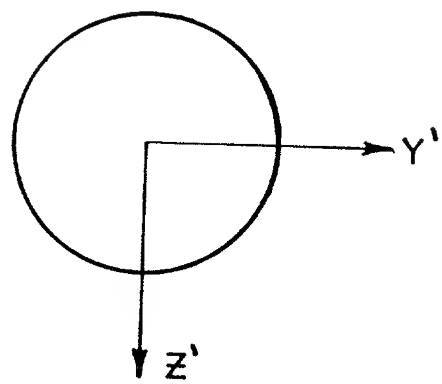
For **Example 2** the equivalent static loading, $p_e(x) = 4.11$, is one-quarter of the loading for Example 1. Consequently, the displacement $v_s = 0.06$ ft. (one-quarter of that in Example 1) and the member forces are one-quarter of the values given in table 11.

8.1.8B Transverse Earthquake Loading

Step 1: Compute deflection $v_s(x)$ under unit load.

This step is identical for both examples.

Table 11: Elastic Forces Due to Longitudinal Earthquake Motion
Example 1



LONGITUDINAL EARTHQUAKE MOTION

Location	$V_{Y'}$ Longit. Shear (kips)	$M_{Z'Z'}$ Longit. Moment (kip-ft)	$V_{Z'}$ Trans. Shear (kips)	$M_{Y'Y'}$ Trans. Moment (kip-ft)	$P_{X'}$ Axial Force (kips)
Abutment 1	0	0	0	0	106(2)
Bent 2 (per column)	1030	12900	0	0	110
Bent 3 (per column)	1030	12900	0	0	115
Abutment 4	0	0	0	0	92

(1) The local Y' and Z' axes of a column or pier do not necessarily have to coincide with the longitudinal and transverse axes of the bridge. However for a straight bridge with no skewed piers, columns or abutments it is recommended, for simplicity of calculations, that the local Y' axis of the column or pier does coincide with the longitudinal axis of the bridge as shown in this example.

(2) The elastic axial forces at the abutments and bents are determined for the loading condition shown in Figure 77 using the moment distribution method and considering the flexibility of the superstructure. The axial forces listed are those from longitudinal motion only and do not include dead load.

A uniform transverse loading of 1 kip/ft is applied to the bridge as shown in figure 78. The stiffness properties required for this analysis are the lateral stiffness of the deck about the global Y axis (I_y), and the transverse stiffness of each column ($I_{Y'}$) in each three-column bent. For this analysis each abutment is assumed to be rigid and to provide a pinned end restraint to the superstructure. The resulting transverse displacements, $v_s(x)$, are tabulated at the quarter points for each span in table 12.

A computer program with space frame analysis capability was used to calculate these deflections but other appropriate methods of analysis may be used if desired. If the abutment is not rigid, its stiffness may be included in this analysis by using the approach outlined in Chapter 7.

Step 2: Compute integrals α, β, γ

This step is identical for both examples.

Calculate the α, β, and γ factors by evaluating the integrals in equations 23, 24 and 26 as follows:

$$\alpha = \int_{\text{Abut. }1}^{\text{Abut. }4} v_s(x)dx = 1.21 \text{ ft}^2 \tag{46}$$

$$\beta = \int_{\text{Abut. }1}^{\text{Abut. }4} w(x)v_s(x)dx = 24.5 \text{ kip-ft.} \tag{47}$$

$$\gamma = \int_{\text{Abut. }1}^{\text{Abut. }4} w(x)v_s(x)^2dx = 0.096 \text{ kip-ft2} \tag{48}$$

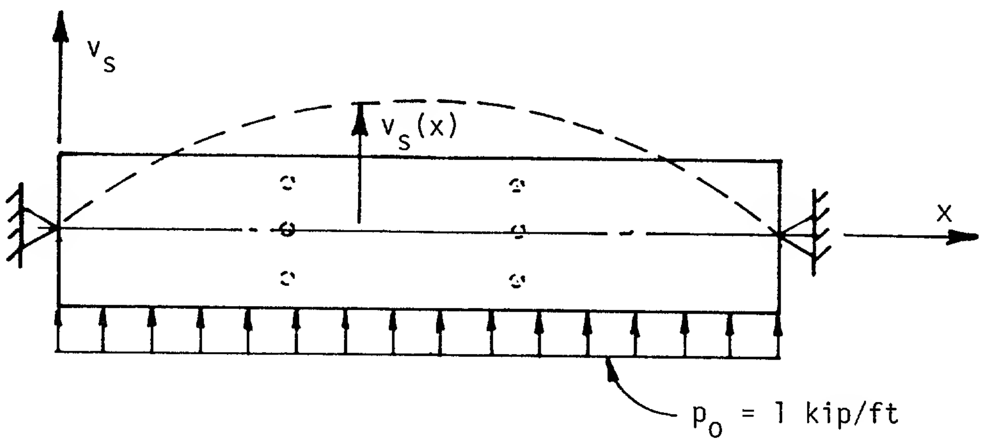
In general the evaluation of the integrals for any structure other than very simple examples will be done using special purpose computer programs (e.g. SEISAB). However, numerical integration may be performed manually by computing the average deflection over segments of the bridge (one-quarter spans in this example) times the length (α), and times the weight (β). To compute γ the square of the average deflections is used.

Step 3: Compute the period, T

This step is identical for both examples.

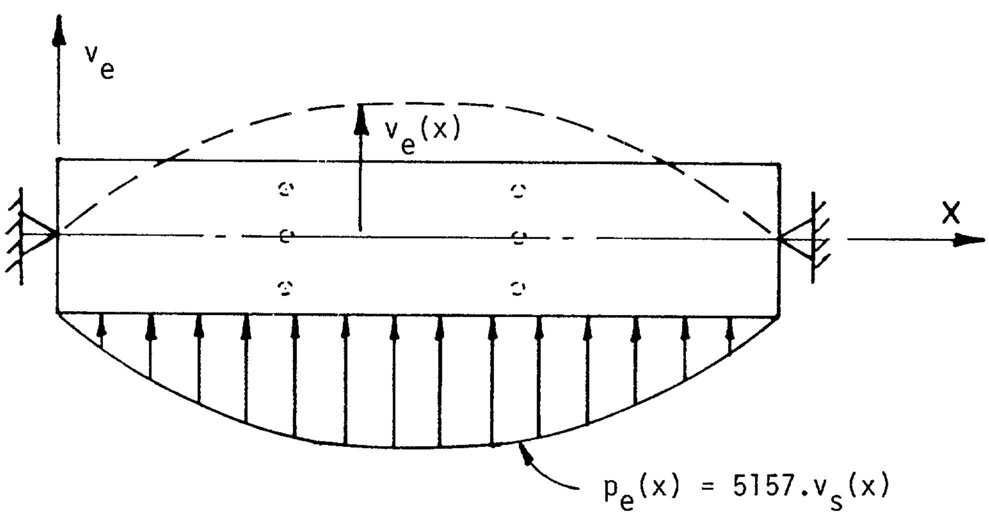
Calculate the period, T, using equation 25 as follows:

$$T = 2\gamma\sqrt{\frac{\pi}{p_o g \alpha}} = 2\pi\sqrt{\frac{0.096}{1.0(32.2)(1.21)}} = 0.314 \text{ sec} \tag{49}$$



Examples 1 and 2

Figure 78: Plan View of Three Span Bridge Subjected to Assumed Transverse Loading (from Reference 4)



Example 1

Figure 79: Plan View of Three Span Bridge Subjected to Equivalent Static Seismic Loading (from Reference 4)

Table 12: Displacements and Seismic Loading Intensity for Transverse Loading : Example 1

Location	Displacements Due to Uniform Transverse Loading $v_s(x)$ (ft)	Seismic Loading Intensity $p_e(x)$ (kips/ft)
Abutment 1	0.0	0.0
Span 1 - 1/4	0.00129	6.66
Span 1 - 1/2	0.00248	12.77
Span 1 - 3/4	0.00348	17.94
Bent 2	0.00425	21.91
Span 2 - 1/4	0.00476	24.54
Span 2 - 1/2	0.00498	25.69
Span 2 - 3/4	0.00490	25.28
Bent 3	0.00453	23.37
Span 3 - 1/4	0.00380	19.58
Span 3 - 1/2	0.00275	14.18
Span 3 - 3/4	0.00145	7.47
Abutment 4	0.0	0.0

$$\alpha = \int v_s(x)dx = 1.21 \text{ ft}^2$$

$$\beta = \int w(x)v_s(x)dx = 24.5 \text{ kip-ft}$$

$$\gamma = \int w(x)v_s(x)^2dx = 0.0965 \text{ kip-ft}^2$$

$$T = 0.314 \text{ sec.}$$

$$p_e(x) = 5157 \; v_s(x) \text{ kips/ft}$$

Step 4: Compute the seismic response coefficient, C_s

The elastic response coefficient, C_s, is obtained from equation 22B. Substituting for A, S and T yields:

Example 1: A = 0.4

$$C_s = \frac{1.2AS}{T^{2/3}} = \frac{1.2 \times 0.4 \times 1.2}{(0.314)^{2/3}} = 1.24 \qquad (50)$$

Example 2 : A = 0.10

$$C_s = 0.31 \qquad (51)$$

These values of C_s are greater than 2.5A for both examples. Therefore use C_s = 1.0 (2.5 x 0.4) for Example 1 and C_s = 0.25 for Example 2, as described in section 6.3. The intensity of seismic loading, $p_e(x)$, is calculated using equation 27. Substituting for β, C_s, w(x) and γ yields:

Example 1: C_s = 1.0

$$p_e(x) = \frac{\beta C_s w(x) v_s(x)}{\gamma}$$

$$= \frac{24.5 \times 1.0 \times 20.3}{0.0965} v_s(x)$$

$$= 5157\ v_s(x)\ \text{kips/ft}^2 \qquad (52)$$

Example 2: C_s = 0.25

$$p_e(x) = 1289\ v_s(x)\ \text{kips/ft}^2 \qquad (53)$$

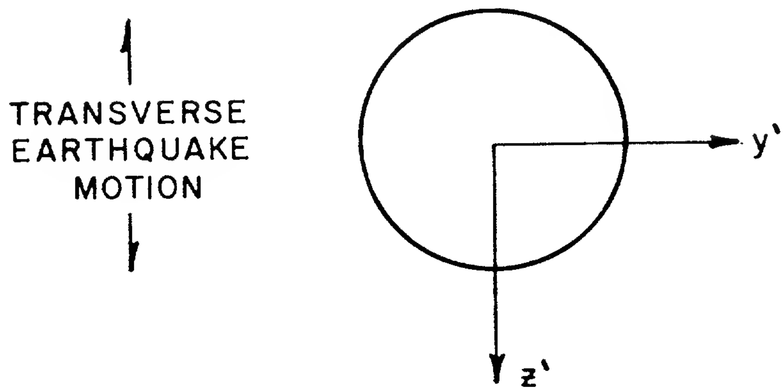
Using this expression, the load intensity at the quarter span points is computed and tabulated for Example 1 in table 12.

For Example 2, the corresponding values of $p_e(x)$ will be one-quarter the values tabulated in table 12. The values of $v_s(x)$, α, β and γ are the same for both examples.

Step 5: Compute forces and displacements under static load

Applying the equivalent static loading as shown in figure 79 yields the member forces due to the transverse earthquake loading shown in table 13. The member forces and displacements in this example were obtained using a computer program with space frame analysis capabilities. Other appropriate methods of analyses can also be used if desired. Note that longitudinal moments and shears, ($M_{Z'Z'}$ and $V_{Y'Y'}$), have been generated by the transverse earthquake because of the flexure of the bridge deck and consequent longitudinal movement of the outer columns of each bent.

Table 13: Elastic Forces Due to Transverse Earthquake Motion Example 1



COLUMN SECTION

Location	$V_{Y'}$ Longit. Shear (kips)	$M_{Z'Z'}$ Longit. Moment (kip-ft)	$V_{Z'}$ Trans. Shear (kips)	$M_{Y'Y'}$ Trans. Moment (kip-ft)	$P_{X'}$ Axial Force (kips)
Abutment 1(1)	0	0	1826	0	0
Bent 2 (per column)	74	887	396	4757	±205(2)
Bent 3(1) (per column)	59	707	424	5089	±219(2)
Abutment 4	0	0	1892	0	0

(1) For this example choose the forces at Abutment 1 and Bent 3 for design purposes.

(2) Axial forces in the bents are in the outermost columns and result from the overturning moment on the bent.

The transverse deck displacements are:

	Bent 2	0.086 ft
Center	Span 2	0.102 ft
	Bent 3	0.092 ft

For Example 2, the corresponding values will be one-quarter of the values given in table 13.

8.1.9 Combination of Orthogonal Seismic Forces

In both examples the combination of forces is the same (section 7.3).

<u>Load Case 1</u> consists of 100% of forces from the longitudinal motion plus 30% of forces from the transverse motion.

<u>Load Case 2</u> has 100% of forces from the transverse motion and 30% of forces from the longitudinal motion.

Table 14 presents the combined forces as given by these two load cases for Example 1 and Table 16 has the corresponding results for Example 2.

8.1.10 Design Forces

Example 1 is in Seismic Performance Category D and is governed by requirements for ductile members capable of forming plastic hinges which apply to Seismic Performance Categories C and D. There are two sets of forces to be determined. The first set is used for the preliminary design of the columns and the second set used to refine the design of the column and the various components connected to the columns.

Example 2 is governed by the Seismic Performance Category B requirements which do not require forces resulting from plastic hinging to be calculated. In some cases it may be economical to perform the plastic hinge calculations but there is no requirement to do so.

8.1.10A Modified Design Forces

These forces are determined in the same way for both examples with the exception of the axial and shear forces in the columns. For all components the combined elastic seismic forces given in table 14 for Example 1 and Table 16 for Example 2 are divided by the appropriate R-Factor before performing the load combinations with dead, live and other appropriate loads. For columns in SPC C and D (Example 1), only the moment is reduced by the R-Factor, the shears and axial forces are not reduced (section 7.6.3).

8.1.10B Design Forces for Structural Members and Connections

The structural members and connections noted in section 7.5.1 which are applicable to both examples are the column members and the abutment shear keys. For design purposes the shear and bending forces at Abutment 1 and Bent 3, were used for each of the load cases tabulated in table 14 for Example 1. For Example 2 the forces are one-quarter the values given in table 14 and are given in table 16. Member dead load forces are shown in table 15 for the critical column in Bent 3 and Abutment 1.

Assume that the earth pressure, buoyancy and stream flow are equal to zero. Using equation 32, the dead load forces tabulated in table 15, and the maximum elastic seismic forces of table 14 divided by the response modification factor where appropriate

Table 14: Maximum Elastic Seismic Forces and Moments for Load Cases 1 and 2 - Example 1

Component	Load Case 1 (1.0 Long. + 0.3 Trans.)	Load Case 2 (1.0 Trans. + 0.3 Long.)
<u>Abutment 1</u>		
$V_{Z'}$-Shear	548 kips	1826 kips
$P_{X'}$-Axial Force	±106 kips*	±32 kips
<u>Bent 3</u>		
$V_{Y'}$-Shear	(1030+18) = 1048 kips	(59+309) = 368 kips
$M_{Z'Z'}$-Moment	(12900+212) = 13112 kip-ft	(707+3870) = 4577 kip-ft
$P_{X'}$-Axial Force	±(115+66) = ±181 kips	±(219+35) = ±254 kips
$V_{Z'}$-Shear	(0+127) = 127 kips	(424+0) = 424 kips
$M_{Y'Y'}$-Moment	(0+1526) = 1526 kip-ft	(5089+0) = 5089 kip-ft

* The axial (i.e., vertical) forces shown were determined using the moment distrubution method as previously stated.

Table 15: Dead Load Forces : Examples 1 and 2

Component	Column (Bent 3)	Abutment 1
$V_{Y'}$-Shear	69 kips	0
$M_{Z'Z'}$-Moment	1170 kip-ft	0
$P_{X'}$-Axial Force	960 kips	624
$V_{Z'}$-Shear	0	0
$M_{Y'Y'}$-Moment	0	0

Table 16: Maximum Elastic Seismic Forces and Moments for Load Cases 1 and 2 – Example 2

Component	Load Case 1 (1.0 Long. + 0.3 Trans.)	Load Case 2 (1.0 Trans. + 0.3 Long.)
Abutments		
V_Z-Shear	137 kips	457 kips
P_X-Axial Force	±26.5 kips	±8 kips
Bents		
$V_{Y'}$-Shear	(258+4) = 262 kips	(15+77) = 92 kips
$M_{Z'Z'}$-Moment	(3225+53) = 3278 kip-ft	(177+968) = 1145 kip-ft
$P_{X'}$-Axial Force	±(29+17) = 46 kips	±(55+9) = 64 kips
$V_{Z'}$-Shear	(0+32) = 32 kips	(106+0) = 106 kips
$M_{Y'Y'}$-Moment	(0+382) = 382 kip-ft	(1272+0) = 1272 kip-ft

The forces and moments shown are one-quarter the values of Table 14.

(see section 8.1.5), the modified design forces are computed as follows.

Modified Design Forces – Columns

By inspection, for **Example 1**, Load Case 1 controls. Note that, in acordance with 8.1.10A above, only the moments and not shears and axial forces are reduced by the R-Factor.

$$V_{Y'}\text{-Shear} = 1.0(D + B + SF + E + EQM) = 1.0\ (69 + 1048) = 1117 \text{ kips} \quad (54)$$

$$M_{Z'Z'}\text{-Moment} = 1.0(1170 + 13112/5) = 3792 \text{ kip-ft}$$

$$P_{X'}\text{-Axial} = 1.0(960 \pm 181) = 779 \text{ or } 1141 \text{ kips}$$

$$V_{Z'}\text{-Shear} = 1.0(0 + 127) = 127 \text{ kips}$$

$$M_{Y'Y'}\text{-Moment} = 1.0(0 + 1526/5) = 305 \text{ kip-ft}$$

Thus for a circular column, the modified design moment is:

$$M = \sqrt{M_{Z'Z'}{}^2 + M_{Y'Y'}{}^2} = 3804 \text{ kip-ft.} \quad (55)$$

By inspection, for **Example 2**, Load Case 1 also controls. For this example, moments shears and axial forces due to earthquake are reduced by the R-Factor.

$$V_{Y'}\text{-Shear} = 1.0(D + B + SF + E + EQM) = 1.0\ (69 + 262/5) = 121 \text{ kips} \quad (56)$$

$$M_{Z'Z'}\text{-Moment} = 1.0(1170 + 3230/5) = 1816 \text{ kip-ft}$$

$$P_{X'}\text{-Axial} = 1.0(960 \pm 46/5) = 951 \text{ or } 969 \text{ kips}$$

$$V_{Z'}\text{-Shear} = 1.0(0 + 32/5) = 6 \text{ kips}$$

$$M_{Y'Y'}\text{-Moment} = 1.0(0 + 382/5) = 76 \text{ kip-ft}$$

Thus for a circular column, the modified design moment is:

$$M = \sqrt{M_{Z'Z'}{}^2 + M_{Y'Y'}{}^2} = 1818 \text{ kip-ft.} \quad (57)$$

Modified Design Forces – Abutment

By inspection for **Example 1** Load Case 2 controls:

$$V_{Z'}\text{-Shear} = 1.0(D + B + SF + E + EQM) = 1.0\ (0 + 1826/0.8) = 2283 \text{ kips} \quad (58)$$

By inspection, for **Example 2** Load Case 2 also controls:

$$V_{Z'}\text{-Shear} = 1.0(D + B + SF + E + EQM) = 1.0\ (0 + 457/0.8) = 571 \text{ kips} \quad (59)$$

After the modified design forces are calculated, the preliminary design of the column as described in Chapter 8 of the AASHTO Guide Specifications, [reference 4] can proceed.

8.1.10C Column Requirements (AASHTO Guide: section 8.4.1)

A column is defined by a ratio of the clear height to maximum plan dimension equal to or greater than 2.5. For these examples, the vertical support has a clear height of approximately 22 ft and a width of 4.0 ft yielding a ratio of 5.5 and thus is classified as a column.

A. Vertical Reinforcement

The vertical reinforcement should not be less than 0.01 or more than 0.06 times the gross area. A ratio not exceeding 0.04 is recommended to minimize placing and congestion problems at splices.

B. Flexural Strength

The modified design forces determined above (section 8.1.10B) are used for the preliminary column design. Considering both the minimum and maximum axial loads the design loads are:

Example 1

$$P = 779 \text{ kips}, \quad M = 3804 \text{ kips-ft} \qquad (60)$$
$$P = 1141 \text{ kips}, \quad M = 3804 \text{ kips-ft}$$

Example 2

$$P = 951 \text{ kips}, \quad M = 1818 \text{ kips-ft} \qquad (61)$$
$$P = 969 \text{ kips}, \quad M = 1818 \text{ kips-ft}$$

The magnification of moment due to slenderness effects is specified in AASHTO Standard Specifications [reference 1], Art. 8.16.5.2 for compression members not braced against sidesway. As specified, the effects of slenderness may be neglected when the slenderness ratio is less than 22. For these columns, the slenderness ratio is slightly greater than 22 and thus slenderness should theoretically be considered. For the purpose of simplicity, however, it has been ignored in these example problems.

For **Example 1**, the column design requires the development of a moment-axial force interaction diagram (figure 80) on a trial and error basis to meet the above requirements. The strength reduction factor (AASHTO Guide: section 8.4.1, part B) shall be 0.50 when the stress due to the axial load exceeds $0.20f'_c$. The value of Φ may be increased linearly to the value for flexure (0.90) when the stress due to the maximum axial load is between $0.20f'_c$ and 0. For this example the maximum column axial stress is $1141/A_c$ where A_c is the core area of the column ($\pi 21^2$ = 1385 sq.in.). Thus the axial stress is 823 psi which is greater than $0.2f'_c$ ($0.2 \times 3250 = 650$ psi) and the Φ factor is therefore 0.50.

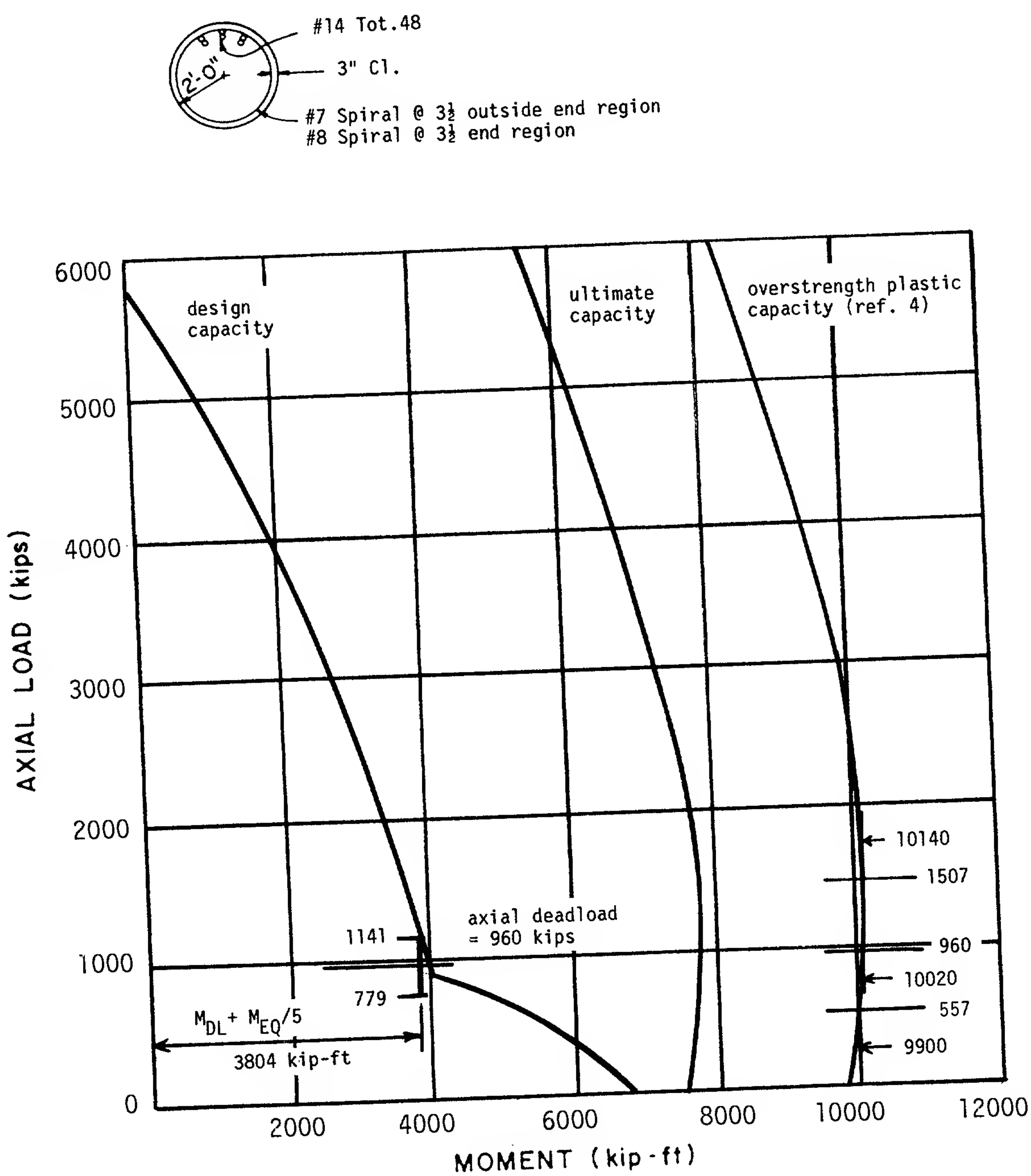


Figure 80: Column Interaction Diagram for Example 1 (after Reference 4)

The column design requires a total of 48 #14 bars, which gives a reinforcement ratio of 0.064, higher than the permitted upper limit. In the same example presented in Appendix A of the AASHTO Guide Specifications only the moment and not the axial load was factored by the Φ factor of 0.5 and consequently 50 #11 bars were used. This example demonstrates that when the factor is correctly applied to both moment and axial load the difference in steel requirement is significant. In fact, in this example a larger column section would be required to satisfy maximum steel requirements.

For **Example 2**, the appropriate strength reduction is also 0.50, since the axial stress is greater than $0.2f'_c$. This strength reduction factor applies to both the moment and axial force on the interaction diagram. For this example, the interaction diagram is shown in figure 81. Note that the Φ factor has been applied to both the axial force and moment.

The column design requires 20 #11 bars of reinforcing steel. This yields a reinforcement ratio of 0.017 for the longitudinal reinforcement which is within the specified limits. A column ultimate capacity interaction diagram along with the reduced design capacity curve is shown in figure 81. The controlling design moment of 1818 kip-ft and axial load are also shown plotted in the figure. The vertical bar indicates the range of axial loads.

This step in the design calculations clearly demonstrates the effect of increased seismic loads in that Example 2 requires less than one-third the amount of vertical reinforcing required for Example 1.

8.1.10D Forces Resulting from Plastic Hinges in Columns

Following the procedure given in section 7.6.2 for bents with two or more columns and using the results of the preliminary design of the column, the forces resulting from plastic hinging may be calculated. Note that this step is required for Example 1 only. The steps required for this calculation are shown in table 17. The column overstrength plastic moment capacity, necessary for this exercise, is included on the interaction diagram shown in figure 80.

8.1.10E Column Design

Given the forces and moments, the design of the column can now be completed.

Example 1

Moment: 3804 kip-ft

Axial Force:

Elastic	960 ± 181 kips
Plastic Hinging	960 ± 547 kips

Shear:

Elastic	$\sqrt{1117^2 + 127^2} = 1124$ kips
Plastic Hinging	922 kips

Note that the forces resulting from plastic hinging for the axial and shear forces are used to complete the column design.

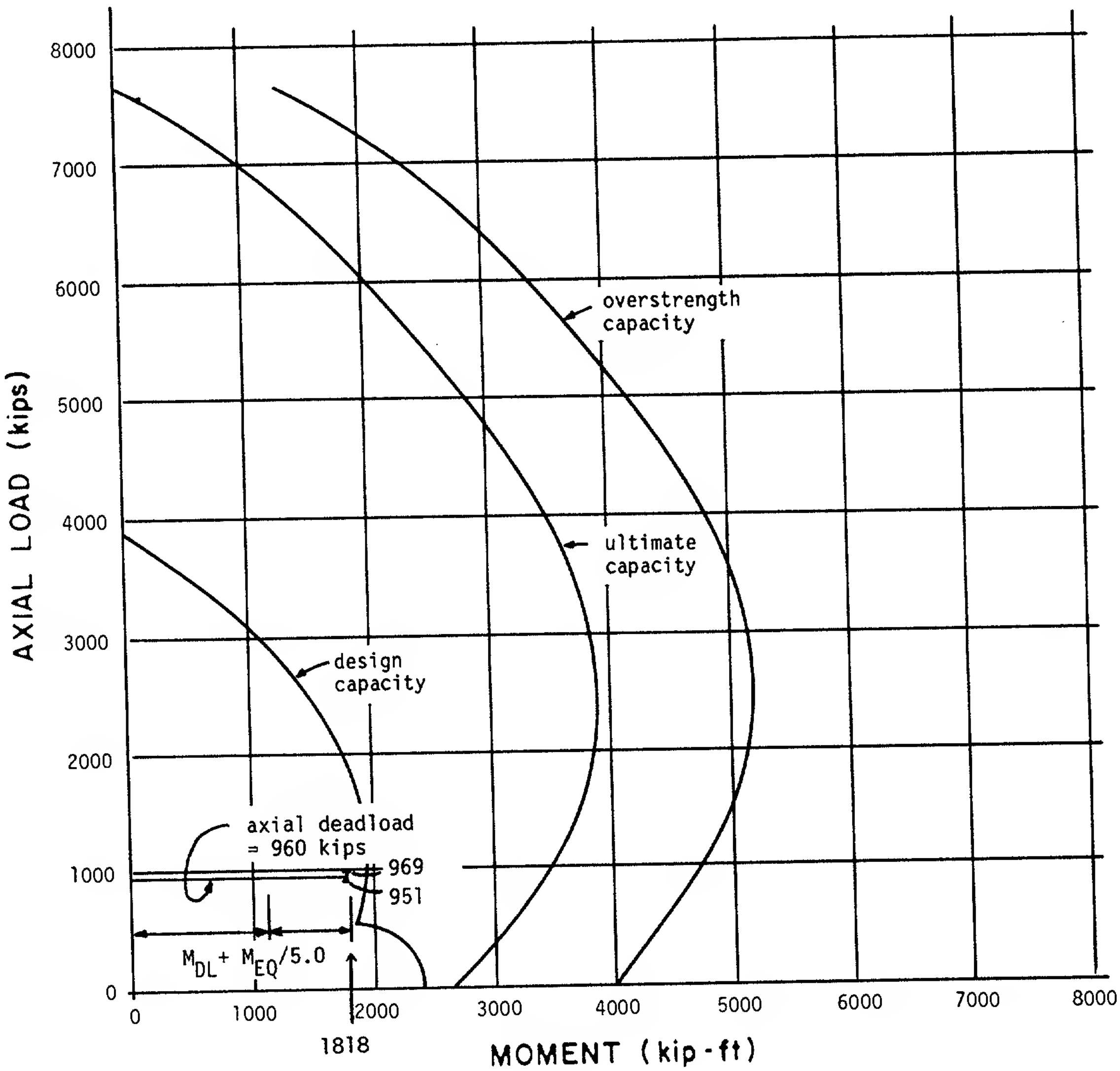
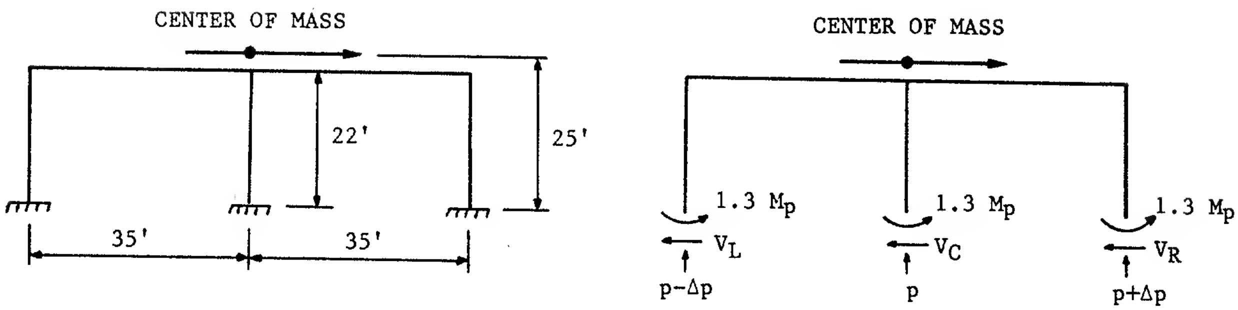


Figure 81: Column Interaction Diagram for Example 2

Table 17: Calculation of Forces Resulting from Plastic Hinging in Columns



Step	1.3 x Mp (kip-ft)			Column Shear Forces (kips)					Column Axial Forces (kips)			% Difference*
	Left	Center	Right	Left	Center	Right	Total	ΔP	Left	Center	Right	
1	10020	10020	10020						960	960	960	
2				912	912	912	2736					
3								548	412	960	1508	--
4	9900	10020	10140	900	912	922	2734					
5								547	413	960	1507	0.2

*Maximum shear force for the bent must be within 10% of previous value as described in Section 7.6.2.B

Example 2

Moment	1818 kip-ft
Axial Force	960 ± 9 kips
Shear	$\sqrt{121^2+6^2}$ = 121 kips

(1). Column Shear and Transverse Reinforcement (AASHTO Guide: section 8.1.4(C))

For **Example 1**, the factored (i.e., plastic hinging) design shear force, V_u obtained above, is 922 kips. Using the strength reduction factor for shear specified in the AASHTO Standard Specifications [reference 1], section 8.16 and equation 8-46 of Art. 8.16.6 of the same Standard Specifications, the factored shear stress for a circular column is:

$$v_u = \frac{V_u}{\Phi bd} = \frac{922}{0.85 \times 48 \times 43} = 526 \text{ psi} \tag{62}$$

The stress carried by the concrete outside the column end regions (AASHTO Standard Specifications, Art. 8.16.6.2) is given by:

$$v_c = 2\sqrt{f'_c} = 114 \text{ psi} \tag{63}$$

Using equation 8-50 of Art. 8.16.6.3 of the same Standard Specifications and the values calculated above for the factored shear stress and the shear stress carried by concrete, the total shear reinforcement A_v is:

$$A_v = \frac{(v_u - v_c)}{f_y} bs = \frac{(526 - 114)}{60{,}000} 48 \times 3.5$$

$$= 1.15 \text{ in}^2 \text{ total area} \tag{64}$$

or $\quad \frac{1.15}{2} \text{ in}^2 = 0.58 \text{ in}^2$ per leg

Therefore, a #7 spiral at 3-1/2 in. pitch should be used outside the column end region.

As **Example 2** is in SPC B, the minimum transverse reinforcement requirements at the top and bottom of a column shall be as required by AASHTO Guide Specification section 8.4.1(D). The spacing of the transverse reinforcement shall be as required in AASHTO Guide Specification section 8.4.1(E), except that the maximum spacing is permitted to increase to 6 inches if not limited by other requirements.

Essentially, the requirements of section 8.4.1(D) of the AASHTO Guide Specification are the same as section 8.18 of the AASHTO Standard Specification where the ratio of the spiral reinforcement shall not be less than:

$$\rho_s = 0.45 \left[\frac{A_g}{A_c} - 1 \right] \frac{f'_c}{f_{yh}} \tag{65}$$

The AASHTO Guide Specification has an additional requirement of:

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \tag{66}$$

The AASHTO Standard Specification, section 8.18 also has a clear spacing requirement between spirals that shall not exceed 3 inches.

$$\rho_s = 0.45 \left[\frac{12.57}{9.62} - 1 \right] \frac{3,250}{60,000} = 0.0075 \tag{67}$$

or

$$\rho_s = 0.12 \frac{3,250}{60,000} = 0.0065 \tag{68}$$

The cross-sectional area of a spiral at 3-1/2 inch pitch is given by:

$$A_{sp} = \frac{\rho_s s D_c}{4} = \frac{0.0075 \times 3.5 \times 41.25}{4} = 0.270 \text{ in}^2 \tag{69}$$

Thus a #5 spiral at 3-1/2 inch pitch should be used over the full height of the column and extend into the top and bottom connections as per section 8.4.3 of the AASHTO Guide Specification.

(2) Column End Region (AASHTO Guide: section 8.4.1(C))

Special requirements for the column end regions and confinement at plastic hinges are required for Example 1 only. The dimensions of the column end region are given by the larger of:

1. Maximum cross-section dimension, d = 4.0 ft
2. One-sixth of clear height, 22/6 = 3.67 ft
3. Eighteen inches

The column cross-section dimension of 4.0 ft is the largest and should be used as the length of the top and bottom end regions. If the minimum axial compression stress is less than $0.1f'_c$ then the concrete shear resistance in the end regions should be neglected. Since

$$\text{minimum axial stress} = \frac{(960-547)}{12.57 \times 144} = 228 \text{ psi} \tag{70}$$

and

$$0.1f'_c = 325 \text{ psi} > 228 \text{ psi} \tag{71}$$

the shear stress taken by the concrete is assumed to be zero. This will yield shear reinforcement, A_v, in the end areas of:

$$A_v = \frac{v_u}{f_y} bs = \frac{526}{60,000} \times 48 \times 3.5 = 1.47 \text{ in}^2 \text{ total area required} \tag{72}$$

or

$$\frac{1.47}{2} \text{ in}^2 = 0.74 \text{ in}^2 \text{ per leg}$$

Thus, a #8 spiral with a 3-1/2 in. pitch in the 4 ft-0 in. end regions at top and bottom of columns should be used.

(3) Transverse Reinforcement for Confinement at Plastic Hinges (AASHTO Guide: section 8.4.1(D))

The volumetric ratio of spiral reinforcement is the greater value given by equation 8-1 or equation 8-2 of Chapter 8 of the AASHTO Guide Specifications. Therefore,

$$\rho_s = 0.45 \left[\frac{A_g}{A_c} - 1 \right] \frac{f'_c}{f_{yh}}$$

$$= 0.45 \left[\frac{12.57}{9.62} - 1 \right] \frac{3250}{60,000} = 0.0075 \tag{73}$$

or

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} = \frac{0.12 \times 3250}{60,000} = 0.0065 \tag{74}$$

The cross-sectional area of a spiral at 3-1/2 in. pitch is given by:

$$A_{sp} = \frac{\rho_s s D_c}{4} = \frac{0.0075 \times 3.5 \times 41.25}{4} = 0.270 \text{ in}^2 \tag{75}$$

Since this is less than the shear reinforcement, there is no additional requirement for confinement at the plastic hinges; thus use #8 spiral at 3-1/2in. in the 4ft-0in. end regions and #7 spiral at 3-1/2 in. throughout the remaining center portion of the column.

8.1.10F Connection Design Forces

Guidelines are given in section 7.6.5 for the design of hold-downs and other connections.

(1) Hold-Down Forces at Abutments

For **Example 1**, hold-down devices are required if the upward reaction due to longitudinal seismic forces exceeds 50% of the dead load reaction (section 7.6.5, part B). The following calculations show that hold-down devices are not required.

Abutment 1

0.5DL = 0.5 x 624 = 312 kips (76)
312 > 106 None Required

For **Example 2** (SPC B) there are no hold-down force requirements.

(2) Column and Pier Connection Design Forces

For **Example 1**, the following design forces which result from plastic hinging (table 17) should be used to design the column connections at the bent cap and the column footings.

Min	Axial	413	kips
	Shear	900	kips
	Moment	9900	kip-ft
Max	Axial	1507	kips
	Shear	922	kips
	Moment	10140	kip-ft

For the connection of the column to pile cap in **Example 2** and to the bent cap the R-Factor is 1 and thus the connection forces are significantly greater than the column design forces.

The following design forces should be used to design the column connections at the bent cap and the column footings.

Axial (960 ± 46) = 914 or 1006 kips (77)
Shear (69 + 262) = 331 kips
Moment (1170 + 3278) = 4448 kip-ft

8.1.10G Foundation Design Forces

For **Example 1**, the following design forces which result from plastic hinging (table 17) should be used to design the foundations at the base of each column. These forces may be applied in any direction. Foundation dead load should be added to the axial forces. (Note that these forces are less than the unfactored elastic forces and so are used for design).

Min	Axial	413	kips
	Shear	900	kips
	Moment	9900	kip-ft
Max	Axial	1507	kips
	Shear	922	kips
	Moment	10140	kip-ft

For **Example 2**, foundation design in SPC B requires the elastic seismic forces divided by half the R-Factor to be used for the substructure design, (section 7.5.2), which in this example is 5/2=2.5.

Thus the following design forces should be used to design the foundations at the base of each column. Foundation dead load should be added to these forces.

$$\begin{aligned} &\text{Axial } (960 \pm 46/2.5) = 942 \text{ or } 978 \text{ kips} \\ &\text{Shear } (69 + 262/2.5) = 174 \text{ kips} \\ &\text{Moment } (1170 + 3230/2.5) = 2462 \text{ kip-ft} \end{aligned} \tag{78}$$

These forces are for load case 1. If the foundation is not symmetrical forces for load case 2 also need to be checked.

8.1.10H Abutment and Retaining Wall Design Force

For **Example 1**, the R-Factor is 0.8 and thus the design forces at Abutment 1 are DL + EQ/0.8 :

$$\begin{aligned} &\text{Vertical loads} \quad 624+106/0.8 = 834 \text{ kips} \\ &\text{Shear-keys} \quad 1826/0.8 = 2283 \text{ kips} \end{aligned} \tag{79}$$

For **Example 2**, the R-Factor is also 0.8 and thus the design forces at abutment 1 are:

$$\begin{aligned} &\text{Axial-bearings} \quad 624 + 26.5/0.8 = 651 \text{ kips} \\ &\text{Shear-keys} \quad 450.5/0.8 = 571 \text{ kips} \end{aligned} \tag{80}$$

8.1.11 Design Displacements

Example 1

The longitudinal displacement at the abutment due to the longitudinal earthquake loading was calculated in Step 5 of section 8.10.1 and is

$$N = 0.239 \text{ ft} = 2.9 \text{ in.} \tag{81}$$

The minimum support length at the abutment bearing seat is calculated from equation 34A (section 7.6.8) as follows:

$$\begin{aligned} N &= 12+0.03L + 0.12H \\ &= 12 + 0.03 \times 376 + 0.12 \times 25 \\ &= 26 \text{ in.} \end{aligned} \tag{82}$$

Thus the support length at the abutments should be at least 26 inches.

Example 2

The longitudinal displacement at the abutment due to the longitudinal earthquake loading was calculated in Step 5 of section 8.10.1 and is:

$$N = 0.06 \text{ ft} = 0.7 \text{ in.} \tag{83}$$

The minimum support length at the abutment bearing seat is calculated from equation 31A (section 7.4.2) as follows:

$$N = 8 + 0.02L + 0.08H$$
$$= 8 + 0.02 \times 376 + 0.08 \times 25$$
$$= 17.5 \text{ inches} \tag{84}$$

Thus the support length at the abutments should be at least 17.5 inches.

8.2 EFFECT OF SUPERSTRUCTURE WEIGHT ON SEISMIC ANALYSIS

The bridge in Examples 1 and 2 is a continuous concrete box girder which is monolithic with the columns and partially restrained at the abutments. An alternate deck configuration of steel or concrete T-girders or a trussed girder would be heavier or lighter depending on construction. To study the effect of variation in superstructure weight, it is assumed that the span configuration is such that the support given to the superstructure is identical to that in Example 1.

For the purpose of illustration, two different superstructure weights are considered--one is half and the other is twice the value used in the detailed example above. These two examples are identified as follows:

Example 3: 3-span continuous girder.
Geometry, piers,abutments and SPC as for Example 1.
Superstructure weight 0.5 x weight of Example 1.

Example 4: 3-span continuous girder.
Geometry, piers,abutments and SPC as for Example 1.
Superstructure weight 2.0 x weight of Example 1.

Note that it has also been assumed that despite the change in weight, these two decks have the same lateral stiffness as that in Examples 1 and 2. Table 18 summarizes the results of the reanalyses for this weight variation. It includes the α, β and γ factors, the period T, the seismic design coefficient C_S and the elastic shear forces at each of the four support locations for both the transverse and longitudinal directions. It is seen in table 18 that:

- A decrease in superstructure weight by a factor of 0.5 shortens the longitudinal period from 0.60 to 0.43 seconds.
- C_S therefore increases from 0.81 to 1.00.
- Column shears decrease by a factor of 0.62 despite the increase in response coefficient. This is because the reduction in superstructure weight is more significant than the increase in the lateral load coefficient.

When the superstructure weight is increased, table 18 also shows that:

- An increase in the superstructure weight by a factor 2 increases the longitudinal period from 0.60 to 0.85 secs.

Table 18: Results of Variations in Superstructure Weight

DIRN.	EX. NO.	MASS RATIO	α (ft^2)	β (k-ft)	γ (k-ft^2)	T (sec)	C_s	$P_e(x)$	SHEARS*			
									A_1 (k)	C_2 (k)	C_3 (k)	A_4 (k)
Trans.	1	1	1.21	24.5	0.096	0.31	1.00	$5157\, v_s(x)$	1830	1190	1270	1890
Trans.	3	1/2	1.21	12.25	0.048	0.22	1.00	$2578\, v_s(x)$	915	595	635	945
Trans.	4	2	1.21	49.0	0.192	0.44	1.00	$10314\, v_s(x)$	3660	2380	2540	3780
Long.	1	1	5.46	110.9	1.61	0.60	0.81	16.45	0	3090	3090	0
Long.	3	1/2	5.46	55.5	0.80	0.43	1.00	10.19	0	1920	1920	0
Long.	4	2	5.46	221.8	3.22	0.85	0.64	26.00	0	4890	4890	0

*Shear in C_2 and C_3 are given per bent, not per column.

Table 19: Results of Variations in Bearing Fixity

DIRN.	EX. NO.	MASS RATIO	α (ft^2)	β (k-ft)	γ (k-ft^2)	T (sec)	C_s	$P_e(x)$	SHEARS*			
									A_1 (k)	C_2 (k)	C_3 (k)	A_4 (k)
Trans.	5	1	1.21	24.5	0.096	0.31	1.00	$5157\, v_s(x)$	1830	1190	1270	1890
Trans.	6	1	2.18	44.3	0.32	0.42	1.00	$2810\, v_s(x)$	3100	0	0	3100
Trans.	7	1	6.49	131.8	2.35	0.67	0.76	$865\, v_s(x)$	0	2040	2050	0
Long.	8	1	5.46	110.9	1.61	0.60	0.81	16.45	0	3090	3090	0
Long.	9	1	11.04	224.1	6.58	0.86	0.64	12.98	0	4880	0	0

*Shear in C_2 and C_3 are given per bent, not per column.

- C_s decreases from 0.81 to 0.64.

- The column shears increase by a factor of 1.58, despite the opposite trend in response coefficient. This is again because the change in superstructure weight is more significant than the variation in load coefficient.

In summary, an increase or decrease in superstructure weight will have a corresponding impact on the forces in the substructure. If the bridge is reasonably stiff with a period that corresponds to the flat part of the design spectra, where changes in C_s are not dramatic (figure 60), then the reduction or increase in substructure forces will be directly proportional to the reduction or increase in the superstructure weight.

8.3 EFFECT OF BEARING CONFIGURATION ON TRANSVERSE RESPONSE

If the continuous superstructure in Example 1 is not cast monolithic with the columns, bearings will be required on the bent caps at the two bent locations and, in all likelihood, at the abutments. With the introduction of bearings, a number of different configurations may be considered. Bearing hardware is such that different restraint conditions may be provided in the longitudinal and transverse directions at each bearing location. These in turn affect the longitudinal and transverse response of the bridge.

In order to illustrate the impact of different fixity conditions in the transverse direction, the superstructure is assumed to be continuous (as in the Example 1) and a relatively stiff bent cap is assumed to connect the three columns in each bent such that the clear height to the bottom of the bent cap is 25 ft. Three different transverse bearing configurations are considered as follows:

Example 5: 3 span continuous bridge.
Bearings at each bent cap and abutment.
Geometry, piers, abutments, weight and SPC as for Example 1.
Pinned transverse connections at abutments.
Fixed transverse connections at columns.
(Same transverse support as for Example 1).

Example 6: 3 span continuous bridge.
Bearings at each bent cap and abutment.
Geometry, piers, abutments, weight and SPC as for Example 1.
Pinned transverse connections at abutments.
No transverse connections at column bearing locations.

Example 7: 3 span continuous bridge.
Bearings at each bent cap and abutment.
Geometry, piers, abutments, weight and SPC as for Example 1.
No transverse restraint at abutments.
Fixed transverse restraint at column bearing locations.

The analytical results for these three configurations are given in table 19. The global distribution of the shear forces to the substructures follows directly from the restraints:

- Example 5 shows shear forces at all 4 supports.
- Examples 6 and 7 show shear forces at either the abutments or the columns respectively.
- For Example 7, the total seismic shear force is reduced from 6200 kips (Example 5) to 4100 kips because of the longer period, and the shear forces in the columns are almost double those of Example 5 because of the lack of restraint at the abutments.

Due to the improved distribution of load to the foundations, transverse restraint at all bearing locations is usually preferable unless there are unusual circumstances such as very weak columns or very weak abutments, which may then require the use of the bearing configurations assumed in Example 6 or 7. A similar and possibly more desirable configuration is the use of elastomeric or isolation bearings at all supports. Not only will a uniform distribution of loads at all four supports be achieved, but also the total and individual shear forces may be reduced very significantly (section 4.7).

8.4 EFFECT OF BEARING CONFIGURATION ON LONGITUDINAL RESPONSE

As in section 8.3, if the superstructure is assumed to be continuous and supported on bearings rather than cast monolithic with the columns, many different bearing configurations are possible. The effect of changing the bearing restraints in the longitudinal direction is illustrated in this section. The two examples considered are:

Example 8: 3 span continuous bridge.
Bearings at each bent cap and abutment.
Geometry, piers, abutments, weight and SPC as for Example 1.
Pinned longitudinal connections at piers.
No longitudinal restraint at abutments.
(Same longitudinal support as for Example 1).

Example 9: 3 span continuous bridge.
Bearings at each bent cap and abutment.
Geometry, piers, abutments, weight and SPC as for Example 1.
Pinned longitudinal connections at one bent location.
Longitudinal expansion joints at all other locations
(i.e. no restraint and free to slide).

The results for these two configurations are also given in table 19. In Example 8, the shear forces are shared equally by the two bents whereas, in Example 9, one bent resists all the shear forces. Although the total shear force is reduced from 6180 kips (Example 8) to 4880 kips due to the longer period, the bent which resists this force (in Example 9) has a 60 percent increase in force demand. If the abutment stiffness had been included in Example 8, the distribution of forces would have been improved and be similar to that calculated for the equivalent configuration in the transverse direction (Example 5).

As with the transverse direction, if elastomeric or isolation bearings are used at all four supports, then not only will a more uniform distribution of forces be found but also the total and individual shear forces may be reduced very significantly.

CHAPTER 9 RETROFITTING

It has become apparent in recent years that many existing bridges in the United States are inadequate to resist seismic loads. Several bridge failures have occured in Alaska and California as a result of seismic activity and some of these have occured at relatively low levels of ground motion. To avoid earthquake related failures in the future, it is clear that an effort must be made to identify seismically deficient bridges and initiate a program for reducing the risk of seismic failure.

The guidelines presented in this chapter are based on the only known comprehensive set of Seismic Retrofit Guidelines for Bridges available in the world. These were developed by the Applied Technology Council with funding from the Federal Highway Administration, and will be referred to in this chapter as the Retrofit Guidelines [reference 5]. These Guidelines are basically an extension of the AASHTO Guide Specification for Seismic Design [reference 4] for new bridges described in earlier chapters. Many of the principles and analysis procedures are the same as those required for new bridges, and where new procedures have been developed, these have been made compatible with the AASHTO Guide Specifications wherever possible.

The seismic retrofitting process can be divided into 3 major steps:

- Preliminary Screening Methodology – to rank or prioritize a large number of bridges in order of decreasing importance taking into account structural vulnerability, lifeline dependency, traffic volume and other relevant issues.

- Detailed Evaluation Procedure – to determine the components of a bridge that require retrofitting.

- Selection and Design of Retrofit Measures.

An overview of this process is provided in section 9.1. The preliminary screening methodology is described in detail in section 9.2. The detailed evaluation procedures are presented and the capacity/demand ratio is introduced in sections 9.3 and 9.4 respectively. Seismic retrofit concepts and their design requirements are discussed in section 9.5.

9.1 OVERVIEW OF THE PROCESS

The Retrofit Guidelines do not prescribe rigid requirements dictating when and how bridges are to be retrofitted. The decision to retrofit a bridge depends on a number of factors, several of which are outside the realm of engineering. These would include, but not be limited to, the availability of funding as well as political, social, and economic considerations. The Retrofit Guidelines assist in evaluating the engineering factors

and in deciding the relative importance of seismic retrofit as against retrofit for other in-service conditions, such as for vehicle or wind loads.

Seismic retrofitting of bridges is a relatively new concept. Only a few retrofitting schemes have been used in practice. At the present stage of development, seismic retrofitting is an art requiring considerable engineering judgment. The Retrofit Guidelines present concepts in seismic retrofitting, but should not be interpreted as restricting innovative designs which are consistent with the principles of good structural engineering.

The primary goal of seismic retrofitting is to minimize the risk of unacceptable damage during a design earthquake. Damage is unacceptable if it results in:

- The collapse of all or part of the bridge.
- The loss of use of a vital transportation route which may pass over or under the bridge.

Because of the difficulty and cost involved in strengthening an existing bridge to new design standards, it is usually not economically justifiable to do so. For this reason, the goal of retrofitting is limited to preventing unacceptable collapse modes of failure while permitting a considerable amount of structural damage during a major earthquake. In some bridges, the ability of the bridge to carry light emergency traffic immediately following an earthquake is also important. The threshold of damage that will constitute unacceptable failure must be defined by the engineer by taking into consideration the overall configuration of the structure, the importance of the structure as a lifeline following a major earthquake, the ease with which certain types of damage can be quickly repaired, and the relationship of the bridge to other structures that may or may not be affected during the same earthquake. Because of the complexity of these decisions and the many nonengineering factors involved, a considerable amount of judgement will be required.

9.1.1 Applicability

The Retrofit Guidelines are intended for use on highway bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 500 ft. This includes approximately 85 to 95 percent of the existing highway bridges. Suspension bridges, cable-stayed bridges, arch-type, and movable bridges are not covered. However, many of the concepts discussed below can be applied to these types of structures if appropriate judgement is used.

The Retrofit Guidelines are recommended for all applicable bridge structures classified as Seismic Performance Category (SPC) B or greater. Seismic retrofit should always be considered when nonseismic rehabilitation is being undertaken or when a bridge is being widened so that the new and old parts of the structure have similar seismic performance. Bridges in SPC A generally do not need to be considered for seismic retrofitting. Minimum requirements for evaluation and upgrading will vary based on the Seismic Performance Category of the bridge.

Preliminary screening is optional for bridges classified in SPC B. However, seismic retrofitting of bridges in this Category should definitely be considered for bridges undergoing non-seismic rehabilitation. These guidelines require that only the bearings, joint restrainers, and minimum support lengths be considered for the retrofit of bridges in Category B.

Bridges in SPC C and D may be subject to the highest potential force levels during an earthquake. Because many bridges were constructed prior to modern seismic design standards, there is a great risk that these bridges will sustain unacceptable damage. Even though current practice has only considered the retrofit of bearings and/or expansion joints, these guidelines propose a methodology whereby all critical components can be evaluated in detail and thus considered for retrofit. This will be increasingly important as more experience is gained and economical methods are developed for retrofitting these other components.

9.1.2 The Retrofitting Process

Not all bridges in the highway system can be retrofitted simultaneously, the most critical bridges should be retrofitted first. The selection of bridges for retrofitting requires an appreciation for the economic, social, administrative, and practical aspects of the problem, as well as the engineering aspects. Seismic retrofitting is only one of several possible courses of action. Others include bridge closure, bridge replacement, or acceptance of the risk of seismic damage. Bridge closure or replacement are usually not justified by seismic deficiency alone and will generally only be considered when other deficiencies exist. Therefore, for all practical purposes a choice must be made between retrofitting or accepting the seismic risk. This choice will depend on the importance of the bridge and on the cost and effectiveness of retrofitting.

The seismic retrofitting process can be divided into three major steps. These are:

- Preliminary screening.
- Detailed evaluation.
- Design of retrofit measures.

A flow chart of the retrofitting process as it applies to bridges in different Seismic Performance Categories is shown in figure 82. Preliminary screening of seismically deficient bridges is necessary to identify bridges which are potentially in the greatest need of retrofitting. This is particularly important when a comprehensive retrofitting program is to be implemented. Certain elements of the screening procedure may also be used to quickly determine if seismic deficiencies exist in individual bridges. The detailed seismic evaluation for retrofitting begins with a quantitative evaluation of individual bridge components and failure modes. The forces and displacements resulting from an analysis of the bridge using the design earthquake are known as **demands**, and these are compared with the ability or **capacities** of the components to resist those forces and displacements. To facilitate this comparison, a component capacity/demand (C/D) ratio is calculated. This ratio is defined and used in a similar manner to a bridge rating factor which may be used in the vertical load capacity evaluation of an existing bridge.

A C/D ratio less than one indicates that component failure may occur during the design earthquake and retrofitting may be appropriate.

An overall assessment of the consequences of local component failure will be necessary to determine the need for retrofitting. Retrofitting should be considered when an assessment indicates that local component failure will result in unacceptable overall performance. The effect of potential retrofitting measures should be assessed by performing a detailed re-evaluation of the retrofitted bridge, since the upgrading of

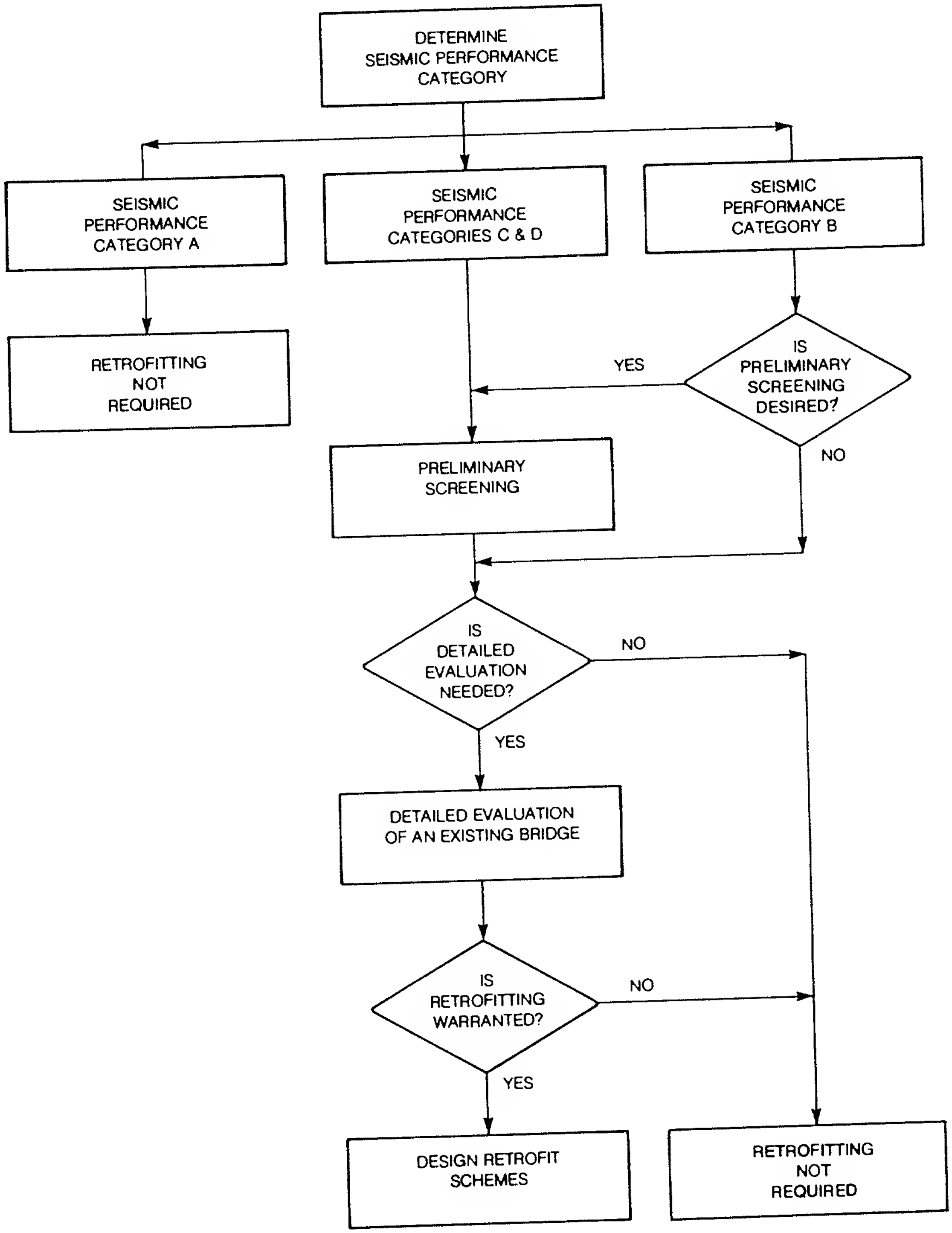


Figure 82: Seismic Retrofitting Process (from Reference 5)

a vulnerable component (e.g. a bearing) may make another component (e.g. a column) more vulnerable than previously assessed.

9.2 PRELIMINARY SCREENING METHODOLOGY

A State or local authority with a significant number of bridges under its jurisdiction may wish to prioritize or rank the bridges in accordance with their need for seismic retrofitting. Section 2 of the Retrofit Guidelines [reference 5] provides a preliminary screening procedure to facilitate the ranking process. A summary of the procedure is provided below. Some of the more refined details for determining component vulnerability are omitted but these can be found in the Commentary for section 2 of the above Guidelines.

An efficient and comprehensive retrofit program requires that structures can be rated according to their need for seismic retrofitting by a preliminary screening process using a seismic rating system. It is recommended that this be done for all bridges classified as Seismic Performance Category C and D. Establishing priorities for retrofitting is optional and greatly simplified for bridges in Seismic Performance Category B. The flow chart shown in figure 83 illustrates the preliminary screening procedure as it applies to bridges in different Seismic Performance Categories.

9.2.1 Seismic Inventory Of Bridges

The first step in implementing the Seismic Rating System is to make an inventory of all applicable bridges with the objective of establishing the following basic information:

- Structural characteristics needed to determine the vulnerability rating described in section 9.2.2A.
- Seismicity of the bridge site.
- Importance of the structure as a vital transportation link.

Preliminary screening of seismically vulnerable bridges should be carried out efficiently and with a minimum of effort. The first step in this process is to accumulate critical information about each applicable bridge on the highway system. The form shown in figure 84 is suggested as one possible means of collecting and recording this information. This completed form and the results of the seismic rating should be included with the existing bridge records.

9.2.2 Seismic Rating System

Although numerical ratings based on a few selected parameters are rarely a totally satisfactory means for determining the priority of needs, they provide a systematic way of considering the major variables involved in any decision. In the case of seismic retrofitting of bridges, there are three major variables that should be considered. These include the vulnerabilility of the structural system, the seismicity of the bridge site, and the importance of the bridge. The proposed Seismic Rating System addresses each of these variables separately by requiring that vulnerability, seismicity, and importance ratings be calculated for each bridge. These individual ratings are combined to arrive at an overall seismic rating. Each of these three areas are assigned a rating

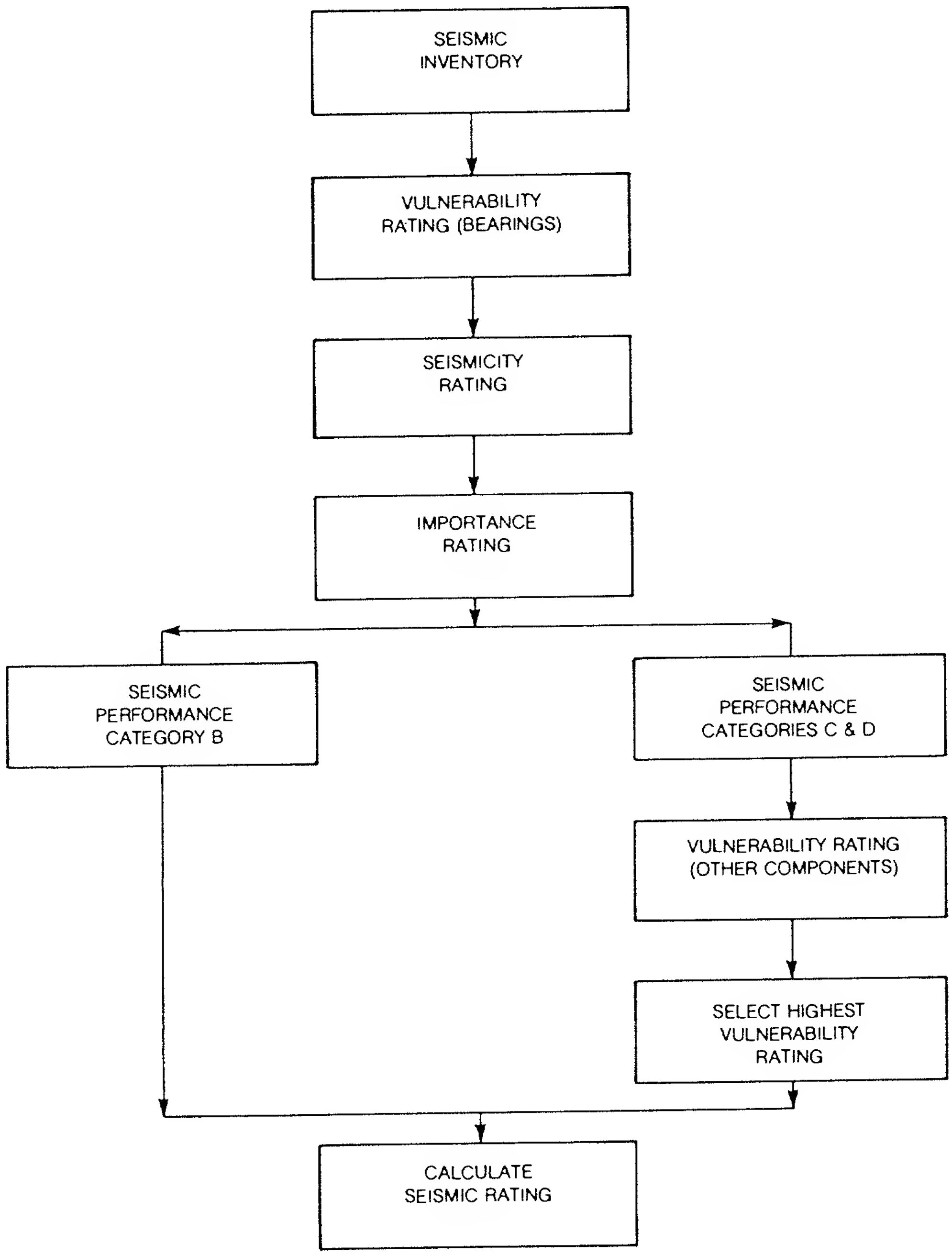


Figure 83: Preliminary Screening Process (from Reference 5)

BRIDGE SEISMIC INVENTORY DATA

GENERAL:

Bridge Name: ______________________________

Location: ______________________________

ADT: ________ Detour Length: ________ Essential Bridge: Yes__ No__

Alignment: Straight____ Skewed____ Curved____ Remarks ____________

Length: ________

Width: ________

Year Built: ________

Seismically Retrofitted: Yes____ No____ Description: ____________

Classification: Regular____ Irregular____ Remarks: ____________

SITE:

Peak Acceleration: ________

Soil Profile Type: I____ II____ III____

Liquefaction Potential: Yes____ No____

SUPERSTRUCTURE:

Material and Type: ______________________________

Number of Spans: ______________________________

Continuous: Yes____ No____ Number of Expansion Joints: ____

BEARINGS:

Type: ______________________________

Condition: Functioning____ Not Functioning____

Type of Restraint (Trans): ______________________________

Type of Restraint (Longit): ______________________________

Actual Support Length: ______ Minimum Required Support Length : ________

Remarks: ______________________________

COLUMNS AND PIERS:

Material and Type: ______________________________

Minimum Transverse Cross-Section Dimension: ____________

Minimum Longitudinal Cross-Section Dimension: ____________

Height Range: _____ Fixity: Top _____ Bottom _____

Percentage of Longitudinal Reinforcement: ____________

Splices in Longitudinal Reinforcement at End Zones: Yes____ No____

Transverse Confinement: ________ Conforms to Design Guideline: Yes____ No____

Foundation Type: ______________________________

ABUTMENTS:

Type: ______________________________

Height: ______________________________

Foundation type: ________ Location: Cut ________ Fill ________

Wingwalls: Continous____ Discontinous____ Length____

Approach Slabs: Yes ____ No _____ Length: ________

SEISMIC RATINGS:

Vulnerability Rating:

Bearings: ________ Other: ______

Highest Rating: ______ Weight: ______ Score: ______

Importance Rating: ______ Weight: ______ Score: ______

Seismicity Rating: ______ Weight: ______ Score: ______

Total Seismic Rating ______

Figure 84: Bridge Seismic Inventory Form (from Reference 5)

and a weight from which a score is calculated. The scores are then added to arrive at an overall seismic rating according to the following procedure:

Vulnerability Rating (rating 0 to 10) x weight	= score
Seismicity Rating (rating 0 to 10) x weight	= score
Importance Rating (rating 0 to 10) x weight	= score
Seismic Rating (100 maximum)	= Total Score

The higher the seismic rating score, the greater the need for the bridge to be evaluated for seismic retrofitting. It is recommended that each weight be taken as 3.33 unless different weights, which must total 10, are assigned by the engineer to reflect regional and jurisdictional needs.

It is obvious that the Seismic Rating System is very subjective. To enhance consistency it is desirable to have the rating of all bridges in one geographical area performed by the same personnel. It is important that the current condition of the bridge be considered in determining these ratings. It is therefore recommended that maintenance personnel who are familiar with the current bridge condition participate in the rating process.

9.2.2A Vulnerability Rating

Vulnerability ratings may assume any value between 0 and 10. In general, a 0 rating means a very low vulnerability to unacceptable seismic damage, a 5 means a moderate vulnerability of collapse or a high vulnerability to loss of access, and a 10 means a high vulnerability to collapse. This does not mean that the vulnerability rating must assume one of these three values. It is useful to consider the seismic vulnerability of the bearings, joint restrainers, and support lengths separate from the vulnerability of the remainder of the structure. The other factors will include columns, piers, footings, abutments, and vulnerability resulting from ground liquefaction. Separate vulnerability ratings between 0 and 10 should be assigned to both of these general areas. The overall vulnerability rating of the bridge shall be taken as the greater of these two ratings, although a record should be kept of both values.

For bridges classified as SPC B, only the vulnerability ratings for bearings, joint restrainers, and support lengths need to be calculated. Determination of these ratings requires considerable judgement. A suggested methodology for determining these ratings is covered in the Commentary for chapter 2 of the Retrofit Guidelines [reference 5].

9.2.2B Seismicity Rating

The seismicity rating shall be taken as 25 times A, where A is the acceleration coefficient taken from the maps in figure 55. The maximum seismicity rating is 10 for an Acceleration Coefficient of 0.4.

9.2.2C Importance Rating

The importance rating will be based on the Importance Classification, IC, of the bridge which is determined from Social/Survival and Security/Defense requirements as discussed in section 6.6. The importance rating may vary from 0 to 10, depending on the relative importance of the structure within each of the Importance Classifications as shown in table 20.

Table 20: Importance Rating

Importance Classification (IC)	Importance Rating
I	6–10 points
II	0–5 points

Since the goal of retrofitting is to minimize unacceptable damage, the relative importance of a bridge is determined by considering the consequences of bridge failure during an earthquake.

Immediate consequences will result from the collapse of the bridge. In this event, the loss of life among individuals on or under the bridge is likely to be high. One factor which will affect the loss of life is the amount of traffic on or under the bridge at the time of the earthquake. This is likely to increase with the amount of traffic that crosses a given point during a given period of time (e.g., average daily traffic) and physical size of the bridge (e.g., length, number of lanes).

Other consequences of failure result from the loss of use of the bridge in the emergency situation that is likely to exist following a large earthqake. This is sometimes very difficult to assess, because there are so many possible situations that may develop in the aftermath of an earthquake. Some of the items that should be considered are discussed in the following section.

9.2.3 Other Factors for Consideration

One very important consideration that is not adequately reflected in the seismic rating system used for preliminary screening is the relationship of the bridge to other bridges on the system that may also be damaged during an earthquake. These types of considerations should be made prior to making a detailed evaluation of the bridge. A few examples will serve to illustrate the influence this consideration has on the decision to retrofit a bridge.

In figure 85, assume that bridge A, has a high seismic rating and is located on a major route in series with lower rated bridges B and C. Assume that no convenient detour to this route exists and that each bridge can be economically retrofitted. The retrofit of only bridge A, the high priority bridge, would only improve one point on the route and do nothing to prevent failure to bridges B and C. In this scenario then, although bridges B and C have lower ratings they both should be considered for retrofit at the same time as bridge A.

The opposite effect could occur if bridge A, in figure 85, had a high rating but can not be economically retrofitted. Because bridge A is in series with bridges B and C, the route would be closed if bridge A were to collapse. Therefore, bridges B and C should be given lower retrofit priority because strengthening of these two bridges without that of bridge A may not prevent closure of the route.

As another illustration, consider two bridges which have parallel functions, such as bridges D and E shown in figure 86. If bridge D is rated at a lower priority than

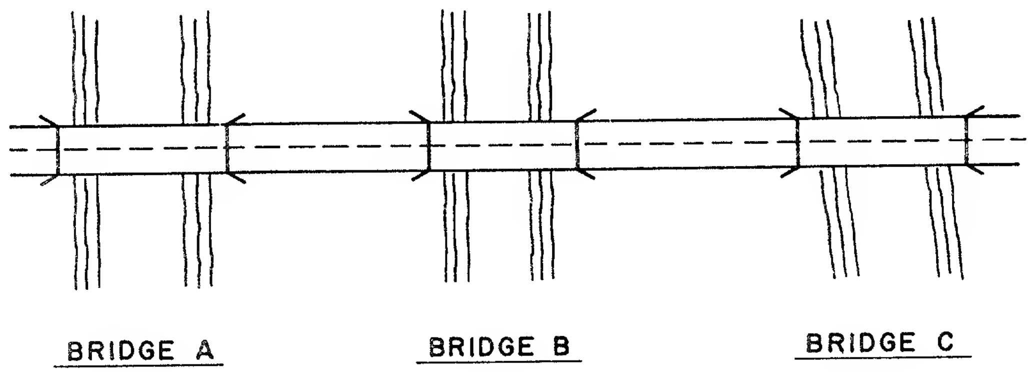


Figure 85: Bridges in Series
(from Reference 5)

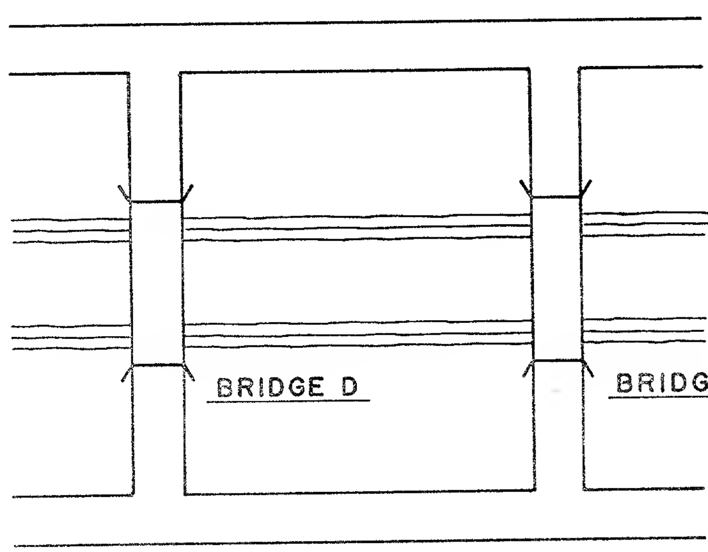


Figure 86: Bridges in Parallel
(from Reference 5)

bridge E, but bridge D is more economical to retrofit, then it might be more rational to retrofit bridge D before bridge E even though bridge E had the higher rating.

A further consideration when a decision to retrofit is to be made is the age and condition of the bridge. It would not be rational to spend a large amount to retrofit a bridge with only five years of service life remaining. However, an unusually high seismic vulnerability may be a justification to accelerate closure or replacement of such a bridge.

A bridge in poor physical condition that is scheduled for nonseismic rehabilitation should be given a higher priority for seismic retrofitting, since construction savings can be realized by performing both the nonseismic and seismic work simultaneously.

9.3 DETAILED EVALUATION PROCEDURE

The detailed seismic evaluation of a bridge will be performed in two phases. The first phase will be a quantitative evaluation of individual bridge components using the results from one of the two analysis procedures specified in the AASHTO Guide Specification [reference 4] and discussed in chapter 7. The analysis will be performed using the design earthquake loading for the site. The resulting forces and displacements (referred to as demands) will be compared with the ultimate force and displacement capacities of each of the components. A capacity/demand ratio is then calculated for each potential mode of failure in the critical components. This ratio is intended to represent the portion of the design earthquake that each of the components is capable of resisting.

The second phase of evaluation is an assessment of the consequences of failure in each of the components with insufficient capacity to resist the design earthquake. Consideration will be given to retrofitting substandard components if their failure results in bridge collapse. In the case of certain essential bridges, the loss of function may also warrant the consideration of retrofitting.

There are four areas where local failure may occur and where component capacity/demand ratios should be calculated. These are:

- Bearings and Expansion Joints.
- Columns, Piers and Footings.
- Abutments.
- Liquefaction of Foundation Soil.

A flow chart detailing this procedure is shown in figure 87, and the calculation of component capacity/demand ratios is given in section 9.4.

9.3.1 Review of Bridge Records

Most agencies maintain a file of as-built bridge plans and a bridge maintenance file with inspection reports and information about major repairs or modifications to each bridge. This information is generally readily available and very useful for the detailed evaluation process. Additional information may also be obtained from the original design calculations and construction records, although these documents are sometimes more difficult to obtain. Bridge rating calculations to determine live-load capacity may also

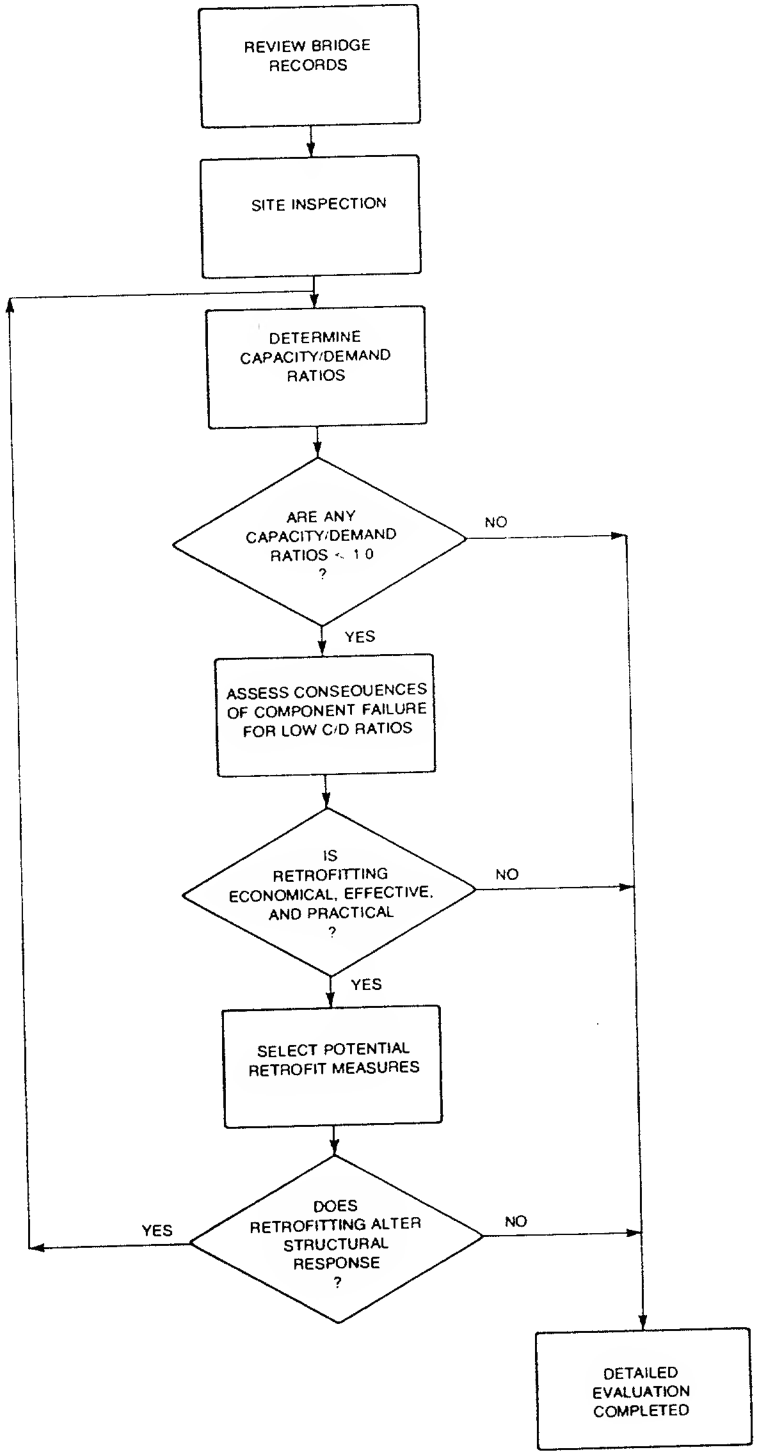


Figure 87: Procedure for Evaluating a Bridge for Seismic Retrofitting (from Reference 5)

contain useful information about the condition and strength of the materials used to construct the bridge.

9.3.2 Site Inspection

Current Federal legislation requires that all bridges over 20 feet in length be inspected biannually as part of the National Bridge Inspection Standards. In general, these inspections are designed to monitor deterioration of the structure, as it may affect the live-load rating, and are not specifically directed toward seismic evaluation. It will, therefore, usually be necessary to make a separate inspection of a bridge to detect seismically vulnerable conditions, or to specifically instruct maintenance personnel to monitor these conditions during their routine maintenance inspections.

A field inspection of bridges selected for detailed evaluation should be made to verify the information obtained from the review of bridge records and to talk to bridge maintenance and inspection personnel. The items which should be noted in the field inspection are as follows:

- Unusual lateral movement under service loading (traffic, temperature, minor earthquake)
- Unusual gap or offset at expansion joints.
- Differential gaps at hinges which result in greatly reduced seat width.
- Large gaps between bridge end diaphragms and abutment backwall.
- Damaged, malfunctioning or unstable bearings.
- Damage or deterioration to the main and secondary structural members.
- Extra dead load, such as wearing surface, utilities, and sidewalks, not shown on plans.
- Unusual erosion of soil at or near the foundation.
- Horizontal or vertical movement or tilting of the abutments, columns, or piers.
- Any deviations from the plans and specifications.

9.3.3 Quantitative Evaluation of Bridge Components

The type of components required to be evaluated for unacceptable failure during an earthquake will vary with the Seismic Performance Category of the bridge. Table 21 indicates the components and failure modes that should be checked in the detailed quantitative evaluation procedure.

Seismic demands will be determined from an elastic analysis of the bridge performed using the design earthquake or from the minimum bearing force and support length requirements that are specified. The limiting available capacity is generally assumed to be one or more of the following:

- The displacement at expansion joints that will result in a total loss of support and collapse of the bridge.

- The ultimate force capacity of fixed bearings.

- The ductile capacity of columns, walls and foundations beyond which unacceptable strength degradation can occur.

Table 21: Components for which Seismic Capacity/Demand Ratios Must be Calculated

Seismic Performance Category	B	C	C	D
Acceleration Coefficient	.09<A<.19	.19<A<.29	.29<A	.29<A
EXPANSION JOINTS AND BEARINGS				
Support Length	X	X	X	X
Forces	X	X	X	X
REINFORCED CONCRETE PIERS AND FOOTINGS				
Anchorage		X	X	X
Splices		X	X	X
Shear		X	X	X
Confinement		X	X	X
Footing Rotation			X	X
ABUTMENTS				
Displacements			X	X
LIQUEFACTION		X	X	X

Table 22: Form for Comparison of Capacity/Demand Ratios

Component	As-Built Bridge	Retrofit Scheme 1	Retrofit Scheme 2
EXPANSION JOINTS AND BEARINGS			
Displacement – r_{bd}			
Force – r_{bf}			
REINFORCED CONCRETE PIERS, AND FOOTINGS			
Anchorage of Longitudinal Reinforcement – r_{ca}			
Splices in Longitudinal Reinforcement – r_{cs}			
Confinement Reinforcement – r_{cc}			
Column Shear – r_{cv}			
Footings – r_{fr}			
ABUTMENTS – r_{ad}			
LIQUEFACTION – r_{sl}			

- Abutment displacements which could result in the bridge becoming inaccessible following an earthquake.

- Foundation movements which are excessive and will result in a collapse of the structure or loss of bridge accessibility.

The basic equation for determining the seismic capacity/demand ratio, r, is:

$$r = \frac{R_C - \Sigma Q_i}{Q_{EQ}} \tag{85}$$

where

R_C = The nominal ultimate displacement or force capacity for the structural component being evaluated.

ΣQ_i = The sum of the displacement or force demands for loads other than earthquake which are included in the group loading defined by equation 32.

Q_{EQ} = The displacement or force demand for design earthquake loading at the site.

Capacity/demand ratios should be calculated at the nominal ultimate capacity without the use of capacity (strength) reduction factors (Φ) so as to obtain a more realistic estimate of the as-built capacity of the members. A more detailed discussion on the calculation of C/D ratios is given in section 9.4.

9.3.4 Identification and Assessment of Potential Retrofit Measures

The capacity/demand ratios indicate the earthquake load levels at which individual components may fail. The capacity/demand ratios for the as-built condition of a bridge should be tabulated as shown in table 22. Values greater than one indicate that the component is not likely to fail during the design earthquake, whereas values less than one indicate a possible failure.

Beginning with the lowest capacity/demand ratio, each value less than one should be investigated to assess the consequences of local component failure on the overall performance of the bridge to identify retrofit measures and to determine the effectiveness of the retrofit measure considered. Component failure is always considered unacceptable if it results in the collapse of the structure. If component failure results in a loss of access or loss of function, this may also be unacceptable if the bridge serves a vital transportation route. If component failure does not result in unacceptable consequences, then retrofitting is usually not justified for the component in question.

If the consequences of component failure are unacceptable, then the effectiveness of retrofitting that particular component should be evaluated. If the response of the remainder of the structure is affected by the retrofit of a component, then new capacity/demand ratios should be calculated and tabulated as shown in table 22. If an improvement in overall bridge performance will result from the component retrofit and this can be accomplished at a reasonable cost, then the bridge should be

retrofitted. Each component with a capacity/demand ratio less than one should be investigated in this way.

9.4 CAPACITY/DEMAND RATIOS

The detailed evaluation of a bridge requires the determination of component capacity/demand ratios as shown in table 21. The demands are obtained either from minimum specified values in the case of bearings and bearing support lengths or from an elastic analysis of the bridge. The analysis procedures and design loads required to calculate the demands are identical to that required for a new bridge, including the combination of forces in orthogonal directions.

9.4.1 Bearings and Expansion Joints

Bridge superstructures are often constructed discontinuously to accomodate anticipated superstructure movements such as those caused by temperature variation or to allow for the use of incompatible materials. Discontinuities necessitate the use of bearings which provide for rotational and/or translational movement. During an earthquake, steel rocker and roller bearings (figure 100) have proved to be among the most vulnerable of all bridge components.

In major earthquakes, the loss of support at bearings has been responsible for several bridge failures. Although many of these failures resulted from permanent ground displacements, several were caused by vibration effects alone. The San Fernando, California earthquake of 1971, the Guatemala earthquake of 1976, and the Eureka, California earthquake of 1980, are some recent examples of earthquakes in which bridge collapse resulted from bearing failure. Even relatively minor earthquakes have caused failure of anchor bolts, keeper bar bolts, welds and nonductile concrete shear keys. In many of these cases the collapse of the superstructure would have occurred if the ground motion were slightly more intense or longer in duration.

Capacity/demand ratios for bearings will be calculated for both displacement and force. Displacements are investigated in the longitudinal direction

The force capacity/demand ratio is calculated for bearings designed to resist lateral loads.

9.4.1A Displacement Capacity/Demand Ratio

The displacement C/D ratios should be calculated for restrained and unrestrained expansion joints and for bearings when movement can occur due to the absence of restraint in a horizontal direction. The displacement C/D ratio is the lesser of the values calculated using the following two methods, except in the case where displacement limiting devices, such as restrainers, are provided, in which case only Method 2 needs to be used.

Method 1:

$$r_{bd} = \frac{N(c)}{N(d)} \tag{86}$$

where

$N(c)$ = The support length provided. This length is measured normal to the expansion joint.

$N(d)$ = The minimum support length as defined in section 7.4.2 or 7.6.8.

Method 2:

$$r_{bd} = \frac{\Delta_s(c) - \Delta_i(d)}{\Delta_{eq}(d)} \tag{87}$$

where

$\Delta_s(c)$ = The allowable movement of the expansion joint or bearing. For structures in SPC D, unreinforced cover concrete should be excluded when determining the allowable movement.

$\Delta_i(d)$ = The maximum possible movement resulting from temperature, shrinkage, and creep shortening. If field measurements have been made of a bridge in existence for some time, only the temperature effects need to be considered.

$\Delta_{eq}(d)$ = The maximum relative displacement due to earthquake loading.

As an example, consider a simply supported bridge with 120-ft span lengths and 30-ft column heights. The minimum support length N(d) for Seismic Performance Category B, from equation 31A, is:

$$\begin{aligned} N(d) &= 8 + 0.02L + 0.08H \\ &= 12.8 \text{ inches} \end{aligned} \tag{88}$$

and for Seismic Performance Category C and D, from equation 34A, it is:

$$\begin{aligned} N(d) &= 12 + 0.03L + 0.12H \\ &= 19.2 \text{ inches} \end{aligned} \tag{89}$$

Thus, if a 6-inch support length is available, then using method 1, the displacement capacity/demand ratio is:

$$\begin{aligned} r_{bd} &= 0.47 \quad \text{for SPC B, and} \\ r_{bd} &= 0.31 \quad \text{for SPC C and D.} \end{aligned} \tag{90}$$

This implies that only 47 percent and 31 percent of the design earthquake can be resisted with the available support length for SPC B and SPC C and D respectively.

9.4.1B Force Capacity/Demand Ratio

The force C/D ratio for bearings and expansion joint restrainers are evaluated as follows:

$$r_{bf} = \frac{V_b(c)}{V_b(d)} \qquad (91)$$

where

$V_b(c)$ = Nominal ultimate capacity of the component in the direction under consideration.

$V_b(d)$ = Seismic force acting on the component. This force is the elastic force determined from an analysis multiplied by 1.25. The minimum bearing force demand of 0.20 DL is used when an analysis is not performed, or when it exceeds the force demand obtained from an analysis.

Elastic bearing forces obtained from a conventional analysis are likely to be lower than those actually experienced by bearings during an earthquake. This is because bearings, which are nonductile components, often do not resist loads in a uniform manner. This has been demonstrated in past earthquakes by the failure of anchor bolts or keeper bars on some, but not all, of the bearings on the same support. In addition, the yielding of ductile members, such as columns, can transfer additional loads to the bearings. For these reasons, it is necessary to increase the elastic forces by a response modification factor, which is less than 1.0, when evaluating the force demand on nonductile motion-restraining components.

The force capacity of bearings must be carefully calculated. Anchor bolts are often subjected to combined bending and shear or high stresses at the threads. Spalling of edge concrete at anchor bolts is also possible. In addition, bearings may not correspond to those shown on the as built plans or maintenance records.

9.4.2 Capacity/Demand Ratios for Reinforced Concrete Columns, Piers, and Footings

It is common for bridge columns to yield during strong seismic shaking. This is expected and provided for in the design of new structures. Existing columns however, may not be capable of withstanding the same degree of yielding as a column designed to a modern code. Failure may also occur prior to yielding in those columns designed to a pre-1971 standard. The interaction of the columns and piers with their footings will determine the probable mode of failure for these components. The first step in their evaluation is to determine if and where plastic hinging will occur. Usually, plastic hinges are found in the end regions of columns or in the footings, but an effect similar to a plastic hinge may also develop due to yielding of the soil or piles. Wall piers can also develop plastic hinges in end regions, but about the weak axis only. The location of plastic hinging will dictate the modes of failure that should be investigated.

Column failures that result in a sudden loss of flexural or shear strength have the potential for causing collapse. The force levels at which these local failures occur

will be reflected in the capacity/demand ratios for the various column failure modes. Each of these modes must be assessed in terms of its effect on the global stability of the structure.

Four modes of failure should be considered when evaluating columns. These are:

- Shear failure in the column.
- Anchorage failure in the main longitudinal reinforcement of reinforced concrete columns.
- Flexural failure in reinforced concrete columns due to inadequate transverse confinement (including bucking failures).
- Failure of the splices in main longitudinal reinforcement of reinforced concrete columns.

Once potential plastic hinges have been located, it is necessary to investigate the potential modes of column and/or footing failure associated with the location and type of plastic hinging. A ductility indicator is used to account for the ability of the columns and/or footings to resist certain modes of failure controlled by the amount of yielding. The ultimate moment capacity/elastic moment demand ratios are multiplied by ductility indicators to enable elastic analysis results to be used for determining the seismic C/D ratios of components subject to yielding.

The following procedure should be used to determine the C/D ratio for columns, piers, and footings as illustrated in the flow chart in figure 88. This procedure includes a systematic method for locating plastic hinges and evaluating the capacity of the columns and/or footing to withstand this plastic hinging. The procedure is more complex for reinforced concrete columns than for steel columns. For steel columns only steps 1, 2, 3 and 6 are required. Sections 4.8.1 through 4.8.5 of the Retrofit Guidelines [reference 5] describe detailed procedures for investigating different reinforced concrete column and/or footing failure modes associated with plastic hinging. A worked example illustrating these procedures is also given in reference 5.

Step 1: Determine the elastic moment demands at both ends of the column or pier for the specified load cases. Moment demands for both the columns and footings should be determined. The elastic moment demand should be taken as the sum of the absolute values of the earthquake and dead load moments.

Step 2: Calculate nominal ultimate moment capacities for both the column and the footings at axial loads equal to the dead load plus, or minus, the seismic axial load resulting from plastic hinging in the columns, piers, or footings. The procedure for calculating this axial load level is discussed in section 7.6.2.

Step 3: Calculate the set of moment C/D ratios (nominal ultimate moment capacity/elastic moment demand), r_{ec} and r_{ef}, for each combination of capacity and demand, assuming, first, that the column will yield and the footing will remain elastic; and, second, that the footing will yield and the column will remain elastic.

Step 4: For reinforced concrete columns, calculate the C/D ratios for the anchorage of longitudinal reinforcement, splices in the longitudinal reinforcement, and/or

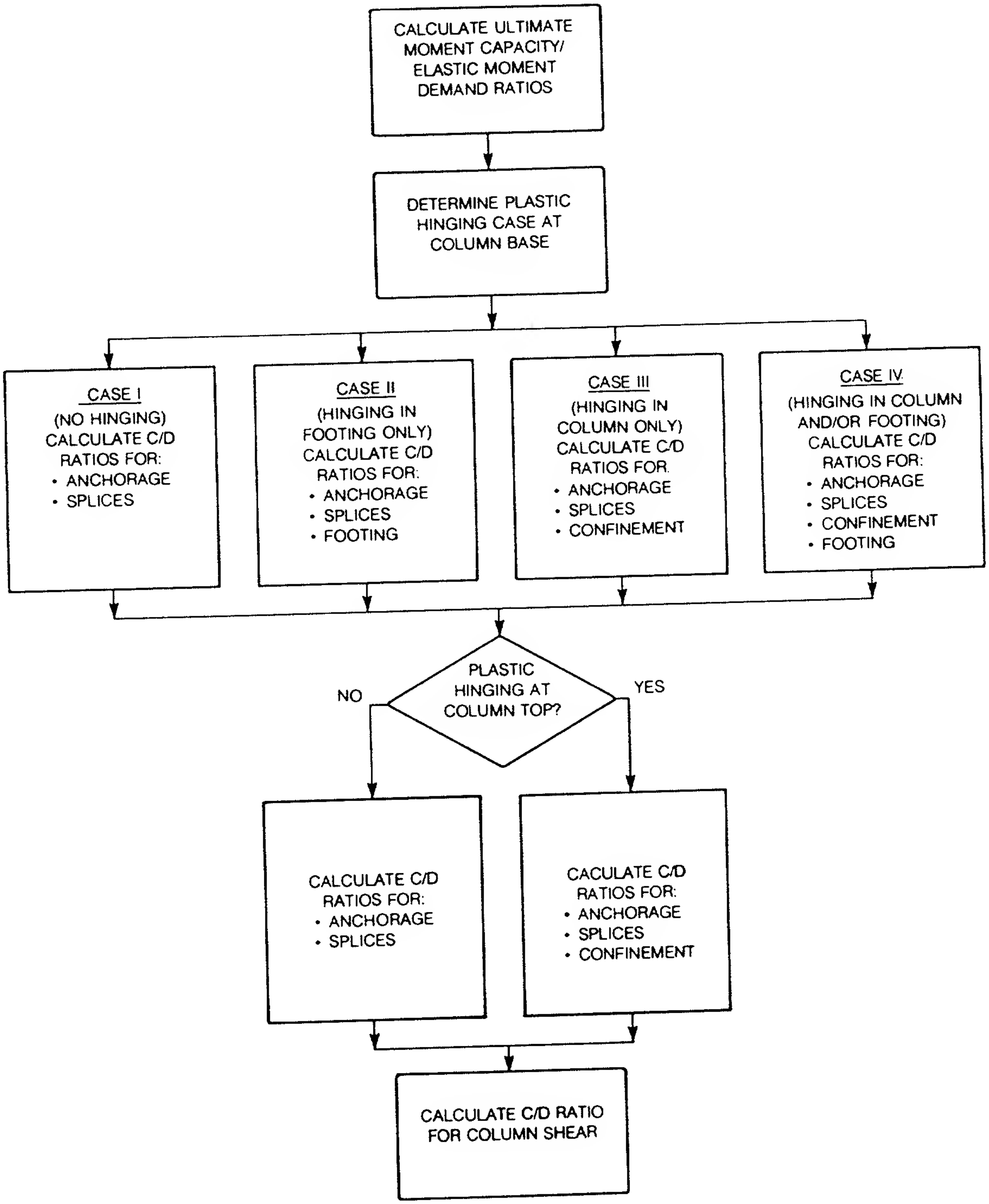


Figure 88: Procedures for Determining Capacity/Demand Ratios for Columns, Piers and Footings (from Reference 5)

transverse confinement reinforcement at the base of the column, and/or footing rotation or yielding for the most severe possible cases of plastic hinging as indicated by each set of r_{ec} and r_{ef}. The following cases describe the C/D ratios that should be investigated based on the location and extent of plastic hinging.

Case I: When both r_{ec} and r_{ef} exceed 0.8, it may be assumed that neither the footing nor the column will yield sufficiently to require an evaluation of their ability to withstand plastic hinging. In this case only the column C/D ratios for anchorage of longitudinal reinforcement and the splices in longitudinal reinforcement should be calculated.

Case II: When r_{ef} is less than 0.8 and r_{ec} either exceeds 0.8 or exceeds r_{ef} by 25 percent, then the footing will require an evaluation for its ability to rotate and/or yield unless an anchorage or splice failure will occur and prevent footing rotation. Anchorage or splice failures may be assumed when either the C/D ratio for anchorage of longitudinal reinforcement or for splices in longitudinal reinforcement is less than 80 percent of r_{ef}. When this is not the case, only the C/D ratio for rotation and/or yielding of the footing should be calculated.

Case III: When r_{ec} is less than 0.8 and r_{ef} either exceeds 0.8 or exceeds r_{ec} by 25 percent, it may be assumed that only the column will yield sufficiently to require an evaluation of its ability to withstand plastic hinging. In this case the column C/D ratios should be calculated for anchorage of longitudinal reinforcement, splices in longitudinal reinforcement, and column transverse confinement.

Case IV: When r_{ec} and r_{ef} are less than 0.8 and within 25 percent of one another, it may be assumed that both the column and footing have the potential to yield sufficiently to require further evaluation. Since yielding of the footings will be prevented by a column failure prior to column yield, column C/D ratios for anchorage of longitudinal reinforcement, splices of longitudinal reinforcement should be calculated first. When all of these C/D ratios exceed 80 percent of r_{ef}, then the C/D ratio for rotation and/or yielding of the footing should also be calculated.

Step 5: For reinforced concrete columns, calculate the column C/D ratios for anchorage of longitudinal reinforcement and splices in longitudinal reinforcement at the top of the column. If the moment C/D ratio, r_{ec}, of the column is less than 0.8, the C/D ratio for column transverse confinement should also be calculated.

Step 6: Calculate the column C/D ratios for column shear.

In reinforced concrete columns, seismic C/D ratios for anchorage of longitudinal reinforcement (r_{ca}), longitudinal reinforcement splice lengths (r_{cs}), column shear capacity (r_{cv}), column confinement reinforcement (r_{cc}), and rotation and/or yielding of the footing (r_{fr}) are dependent on the amount of flexural yielding in the column or footing. In columns with poorly detailed transverse reinforcement, one of the most critical consequences of flexural yielding is the spalling of cover concrete. Such spalling is followed by a rapid degradation in the effectiveness of the transverse steel which can lead to column failure. The procedure for calculating C/D ratios for column confinement reinforcement is based on the assumption that spalling will begin at a

ductility indicator of 2. The effectiveness of poorly detailed transverse reinforcement is assumed to begin to degrade at the onset of spalling. This type of transverse reinforcement is considered totally ineffective beyond a ductility indicator of 5. A more detailed description of the various modes of failure in reinforced concrete columns follows.

9.4.2A Shear Failure

Shear failures in reinforced concrete columns occur suddenly and can result in the rapid disintegration of the column. This happened to several bridges during the San Fernando earthquake (see figures 27, 28, and 29). Flexural yielding of the column has the effect of limiting the shear force, but it also results in a degradation of shear capacity. The guidelines provide techniques for determining the level of yielding at which the danger of a shear failure is large. The level of yielding is represented by a ductility indicator which is applied to the flexural capacity. The capacity/demand ratio for column shear is then determined by comparing the modified flexural capacity with the elastic flexural demand.

If the initial shear capacity is less than the ultimate shear force resulting from flexural yielding of the column, then the seismic capacity/demand ratio will be calculated as the ratio of the initial shear capacity to the elastic shear force caused by the design earthquake. If the final shear capacity is greater than the ultimate shear force resulting from flexural yielding of the column, then shear need not be considered a critical mode of failure. When yielding occurs in the footing, column shear capacity will not deteriorate, and shear failure may occur only if the ultimate shear force exceeds the initial shear capacity.

9.4.2B Anchorage Failure

A sudden loss of flexural strength in reinforced concrete columns can result from an anchorage failure of the main reinforcement. This type of failure occurred at the Route 210/5 Separation and Overhead during the 1971 San Fernando Earthquake (see figures 30 and 31). When cracking occurs in the concrete where reinforcing steel is anchored, bond capacity is lost, and this type of failure is more likely. The procedures for calculating capacity/demand ratios for longitudinal steel anchorage take this into consideration.

9.4.2C Flexural Failure

Sufficient transverse confining reinforcement in reinforced concrete columns is necessary to prevent strength degradation in flexure. In most existing columns the transverse reinforcement is not capable of preventing flexural degradation at the levels of yielding assumed in the design of new columns. Therefore, a method for determining the reduced levels of yielding at which existing columns will fail is proposed in the Retrofit Guidelines. This is also done through the determination of a ductility indicator that is applied to the ultimate flexural capacity of the column. This modified flexural capacity is divided by the elastic moment in the column to obtain the capacity/demand ratio.

The practice of splicing reinforcing bars at the bottom of the column was common in the past and may result in a high potential for failure during an earthquake. Flexural yielding of the column is likely to occur at this location, which will greatly reduce the capacity of the splices. The Guidelines consider this type of failure by limiting

the amount of allowable yielding that can take place at a location where splices occur.

The capacity/demand ratio for the footing in flexure is calculated when yielding occurs in the footing. The allowable amount of flexural yielding will depend on the mode of the footing failure. This is also represented by a ductility indicator that is applied to the ultimate footing flexural capacity.

9.4.3 Capacity/Demand Ratios for Abutments

Failure of abutments during an earthquake usually involves tilting or shifting of the abutment, either due to inertia forces transmitted from the bridge superstructure or to large seismic earth pressures. Usually these types of failures alone do not result in collapse or impairment of the ability of the structure to carry emergency traffic loadings. However, these failures often result in loss of access, which can be critical in certain important structures.

Large horizontal movement at the abutments is often the cause of large approach fill settlements that can prevent access to the bridge. Therefore when required, abutment C/D ratios are based on the horizontal abutment displacement. The displacement demand d(d), will be the elastic displacement at the abutments obtained by properly modeling the abutment stiffness. The displacement capacity, d(c), is taken as three inches in the transverse direction and six inches in the longitudinal direction unless determined otherwise by a more detailed evaluation. Therefore:

$$r_{ad} = \frac{d(c)}{d(d)} \tag{92}$$

9.4.4 Capacity/Demand Ratios for Liquefaction

To determine the C/D ratio for liquefaction failure, r_{sl}, a two-stage procedure is necessary. First, the depth and extent of soil liquefaction required for foundation failure must be assessed. Secondly, the level of seismic shaking that will produce liquefaction of the soils must be evaluated. The C/D ratio is obtained by dividing the effective peak ground acceleration at which liquefaction failure is likely to occur by the design acceleration coefficient:

$$r_{sl} = \frac{A_L(c)}{A_L(d)} \tag{93}$$

where

$A_L(c)$ = The effective peak ground acceleration at which liquefaction failures are likely to occur.

$A_L(d)$ = A = Design acceleration coefficient for the bridge site.

Although a great deal of work has been done with respect to determining earthquake induced liquefaction potential of soils, the parameter $A_L(c)$ will require considerable engineering judgment. The amount of movement at a given site due to soil liquefaction is a function of the intensity and duration of shaking, the extent of liquefaction, and also the relative density of the soil, which controls post-liquefaction undrained or residual strength. In addition, different bridges will be able to sustain different amounts of movement. Therefore, when determining $A_L(c)$, both the site and the bridge characteristics must be taken into consideration.

Methods for assessing the liquefaction potential of site soils are provided in the AASHTO Guide Specifications [reference 4]. Two basic approaches are typically used, namely empirical methods based on blow count correlations for sites which have not liquefied, and analytical techniques based on the laboratory determination of liquefaction strengths and dynamic site response analyses. A rough indication of the potential for liquefaction may be obtained by making use of empirical correlations between earthquake magnitude and epicenter distance as described in reference 4.

Finally, it is recommended that geotechnical specialists participate in the determination of $A_L(c)$ at a specific bridge site and assist in the evaluation of the subsequent foundation displacement and damage potential.

9.5 SEISMIC RETROFITTING CONCEPTS

Seismic retrofitting concepts are designed to prevent collapse and/or severe structural damage of the bridge due to the following modes of failure:

1. Loss of support at the bearings which will result in a partial or total collapse of the bridge.
2. Excessive strength degradation of the supporting components.
3. Abutment and foundation failures resulting in loss of accessibility to the bridge.

Once a concept has been selected, it must be evaluated to ensure that it does not transfer excessive force to other less-easily inspected and repaired components.

Bridges in Seismic Performance Category B will usually only require consideration of measures for the retrofit of the bearings and expansion joints. In Seismic Performance Category C, columns, piers and footings should also be considered. Only in Seismic Performance Category D should the retrofit of all components be considered.

Once it has been decided to retrofit a component, it is recommended that the component retrofit be designed to the standards for new construction, wherever possible. Reduced standards may be used when the use of full design standards is not practical or economically feasible and partial strengthening significantly reduces the risk of unacceptable damage. The following sections provide an overview of some of the retrofit concepts that have either been used or proposed for use. Special design requirements are also presented where necessary.

9.5.1 Bearing and Expansion Joints

Several bridges have failed during past earthquakes due to a loss of support at the bearings. These failures are sometimes spectacular, but are also relatively simple and inexpensive to prevent. Because of this, most retrofitting efforts in the United States to date, have been directed towards tying the bridge together at bearings and expansion joints. Several retrofitting methods have been used extensively, while newer methods such as seismic isolation, have only recently been tried. The methods to be considered for bearing and expansion joints are:

- Longitudinal Joint Restrainers.
- Transverse Bearing Restrainers.
- Vertical Motion Restrainers.
- Bearing Seat Extensions.
- Replacement of Bearings.
- Special Earthquake Resistant Bearings and Devices.

9.5.1A Longitudinal Joint Restrainers

Longitudinal joint restrainers are used extensively by the California Department of Transportation. The primary function of these devices is to limit relative displacements at joints and thus decrease the chances for a loss of support at these locations.

However, if connected to the bent caps, these restrainers may impose higher force levels on the columns than otherwise expected and this possibility should be carefully assessed.

Restrainers are designed to resist forces in the elastic range. Careful attention must be given to the methods used to attach restrainers to the superstructure so that existing components will not be damaged during an earthquake. Provision must also be made for the protection against corrosion, especially for those that cannot be easily inspected, e.g. restrainers which pass through holes cored in existing structural members.

The restrainer force capacity and stiffness will generally be determined from an analysis of the structure. However, results from an analysis should always be carefully examined and interpreted with engineering judgement in light of the several assumptions usually made in a dynamic analysis. When higher forces seem appropriate, they should be used for design. In no case should the restrainer force capacity be less than that required to resist an equivalent horizontal static load of .35 times the dead load of the superstructure. When two superstructure segments are tied together, the minimum restrainer capacity should be the maximum of the two capacities obtained by considering each section independently. For "regular" bridges in Seismic Performance Category B, an analysis is not necessary, and the minimum restrainer force capacity may be used as the restrainer design force. As described in section 7.6.5A, this minimum force is given by the product of the Acceleration Coefficient and the weight of the lighter of the two adjoining spans or parts of the bridge. Restrainers should be capable of developing the design force before the bearings become unseated. A minimum of two symmetric restrainers per joint will provide for redundancy and minimize eccentric movement of the joint. An adequate gap should be provided to allow for normal movement at expansion joints. For joints located at piers, restrainers should provide a direct and positive tie between the superstructure and the pier, unless pier caps are wide enough to prevent a loss of support at the end of the span and the anticipated maximum movement of the superstructure will not cause excessive damage to the bridge.

Connections of the restrainer to the superstructure or substructure should be capable of resisting 125 percent of the ultimate restrainer capacity. In addition, the existing structural elements subject to brittle failure should also be capable of resisting 125 percent of the ultimate restrainer capacity. Both restrainer connections and existing structural elements should be capable of resisting the eccentricities caused by variations in the restrainer forces of at least 10 percent of the nominal ultimate restrainer capacity.

Longitudinal restrainers should be oriented along the principal direction of expected movement. If piers are rigid in the transverse direction, as shown in figure 89, the movement of the superstructure will be along the longitudinal axis of the bridge, and the restrainers should be placed accordingly. However, in a skewed bridge with transversely flexible supports, superstructure rotation can occur. In this case, restrainers will be more effective if placed normal to the expansion joint as shown in figure 90. If damage to the restrainers due to shearing action is a possibility in a predominantly longitudinal event, then transverse shear keys might also be necessary. In this event, the restrainers can be parallel with the bridge centerline as in figure 89.

When an expansion joint exists at a pier, restrainers at the expansion joint should provide a positive tie to the pier, as shown in figure 91. This detail will require that each restrainer resist the inertia forces of both spans. Depending on the configuration of the restrainers at adjacent expansion joints it is possible that the inertia forces of other spans should also be included. Note that in figure 91 the restrainers are connected to the bottom flange. This will prevent the possibility of tearing the web but it will also reduce vertical clearance under the bridge.

In some cases it may be appropriate to forego the positive tie to the pier. Adjacent spans may then be tied, as shown in figure 92. This should be considered only when the cumulative openings of expansion joints is small enough to prevent the spans from becoming unseated, when positive ties could excessively overload the pier and/or when one of the spans has an adequate existing connection to the pier. Although this retrofit technique is unlikely to prevent rocker bearings from toppling, collapse of the span will be prevented by the pier cap, if it has sufficient width. Minor emergency repairs could quickly restore the usefulness of the bridge.

Steel cables and bars acting in direct tension have been the most frequently used method for restraining expansion joints against excessive movements. These devices do not dissipate any significant amount of energy because they are generally designed to remain elastic. Cable and bar restrainers may permit the ends of girders to be damaged, but the damage will usually be repairable and not extensive enough to allow the spans to lose support. Although cables and bars do not meet all the criteria of an ideal restrainer, they are relatively simple to install.

The California Department of Transportation has been retrofitting bridges with longitudinal expansion joint restrainers since the San Fernando earthquake of 1971. They have used two types of restrainer materials. The first type is 3/4-inch diameter galvanized steel wire rope (6 strands with 19 wires per strand) identical to the material commonly used to anchor the ends of barrier railings. The second type of material is 1-1/4" diameter, high strength steel bars. These bars are also galvanized and conform to ASTM A-722 standards. In addition, these bars are required to provide elongation of at least 7 percent in 10 bar diameters before fracture.

Caltrans has no established rule as to when wire rope or bars are preferred. Since restrainers are designed to perform elastically, the extra ductility of the 1-1/4 inch bars is not considered to be a particular advantage. An important consideration is the amount of movement allowed at the expansion joint. Elastic stretching should be limited because excessive movement can result in a loss of support at narrow bearing seats. On the other hand, an overly stiff restrainer, although more effective in limiting movement, will attract higher forces. In California, the results of multimodal spectral

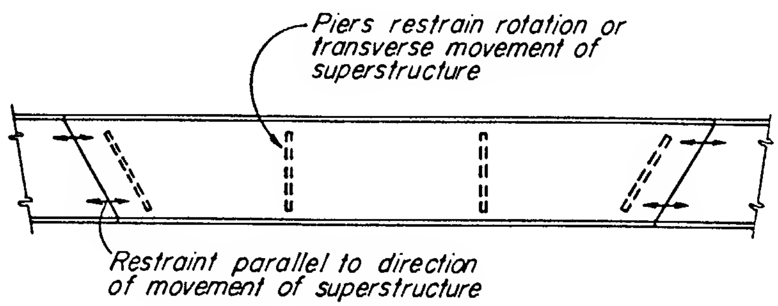


Figure 89: Restrainer Orientation – Transversely Rigid Supports (from Reference 5)

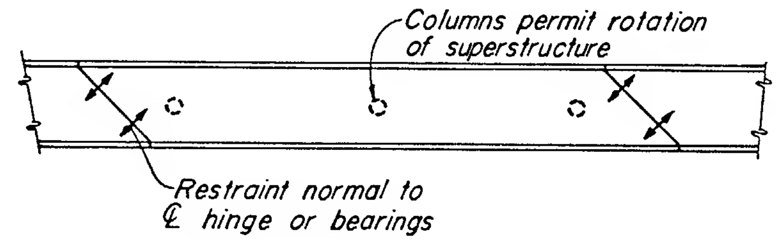


Figure 90: Restrainer Orientation – Transversely Flexible Supports (from Reference 5)

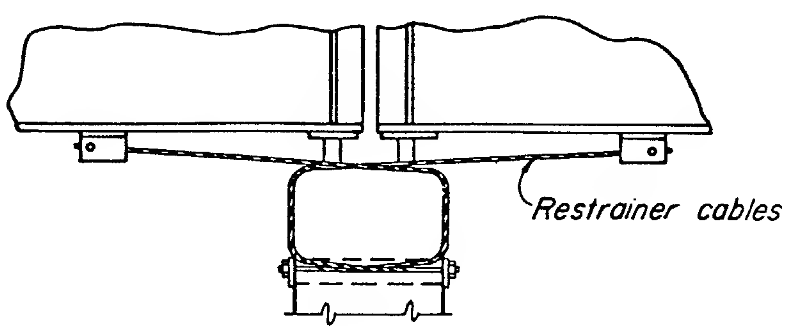


Figure 91: Restrainer at Pier – Positive Tie to Pier (from Reference 5)

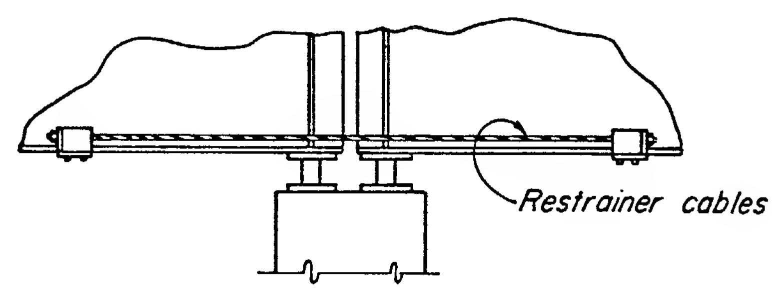


Figure 92: Restrainer at Pier – No Positive Tie to Pier (from Reference 5)

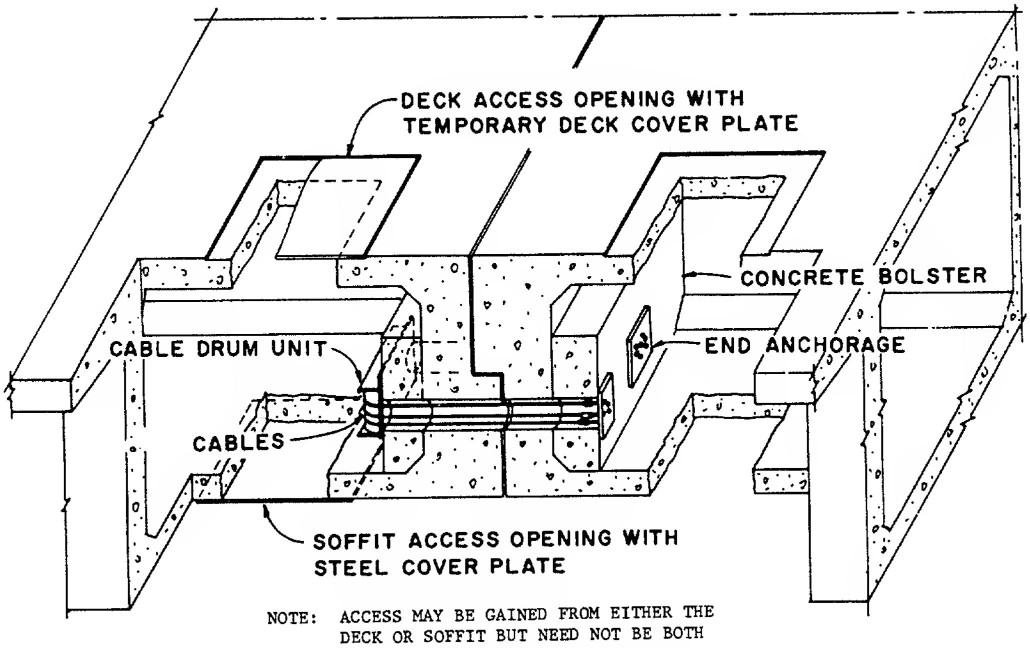


Figure 93: Longitudinal Joint Restrainer for Concrete Box Girder (from Reference 5)

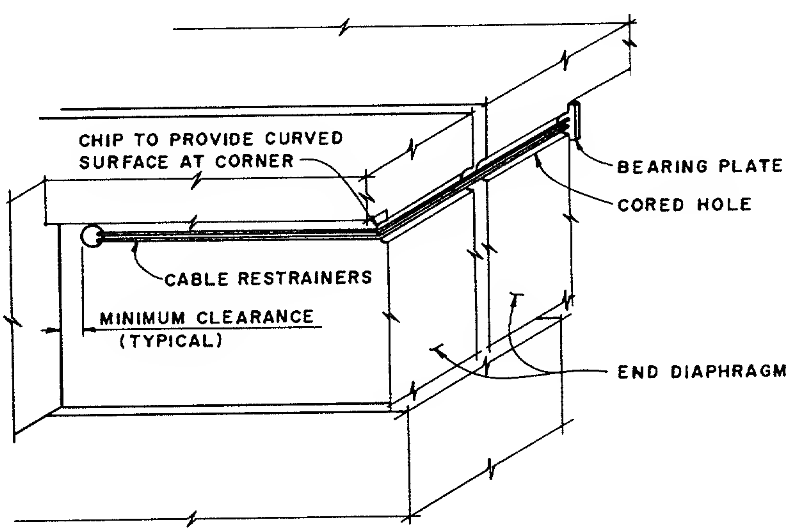


Figure 94: Expansion Joint Retrofit Detail for Concrete T Beam (from Reference 5)

analyses are used to select the right combination of restrainer stiffness and strength. The number and length of wire ropes or bars are then selected on this basis.

Wire ropes often have an economic advantage, since shorter lengths are required to allow for a given amount of movement. In addition, wire ropes are flexible and more able to accomodate transverse and vertical movements. If bars are used, transverse and vertical restrainers may be required to prevent a shear failure in the bars.

Figure 93 shows a method for retrofitting an intermediate expansion joint in a concrete box girder. Either wire rope or rigid steel bars may be used to prevent separation of the joints. Concrete bolsters are sometimes necessary to strengthen the diaphrams to accomodate the force transmitted from the restrainers.

In open web concrete bridges such as "T" beams, the lack of support at the bottom edge of the diaphram may make it necessary to locate restrainers as shown in figure 94. This detail is usually restricted to situations where the restrainer force requirements are relatively low. When the joint is located at a bent, a positive tie between the substructure and the superstructure is preferred to this detail unless the bridge is relatively short and bent caps are wide enough to prevent loss of end support.

An alternate method for restraining joints when the diaphragm is weak, is to attach restrainers to the sides of the girders or to the underside of the deck. In this case, it is necessary to locate restrainer anchors a sufficient distance from the joint to prevent damage to the ends of the span. A detail in which restrainers are anchored to the deck is shown in figure 95. A direct tie to the bent may be difficult when anchoring restrainers in this way.

Certain special situations permit some variation in the use of restrainer details. For example, figure 96 shows continuous wire ropes used to restrain a suspended span. Large restrainer lengths often make it necessary to increase the number of restrainers to limit the relative movement at the joints. Therefore, although anchorage costs are reduced with this detail it may not be economical due to the excessive length required of each restrainer.

Other devices, such as bumper brackets bolted to girder flanges and designed to impact abutments, or bent caps to restrict movement, should also be considered.

9.5.1B Transverse Bearing Restrainers

Transverse restrainers are necessary, in many cases, to keep the superstructure from sliding off the bearings. Conditions that are particularly vulnerable include high concrete pedestals, which serve as bearing seats for individual girders; bearing seats which are narrow and highly skewed; and in two girder bridges in which the transverse distance between the bearing and the edge of the seat is small.

The forces used to design transverse restrainers are generally determined from an analysis. Transverse restrainer forces obtained from the elastic design spectra should be increased by a factor of 1.25 to account for transfer of load due to column yielding. The minimum transverse restrainer design capacity should be not less than that required to resist an equivalent horizontal static load of 0.35 times the superstructure dead load. For single-span bridges or "regular" bridges in Seismic Performance Category B, an analysis is not necessary and the minimum transverse design force may be used.

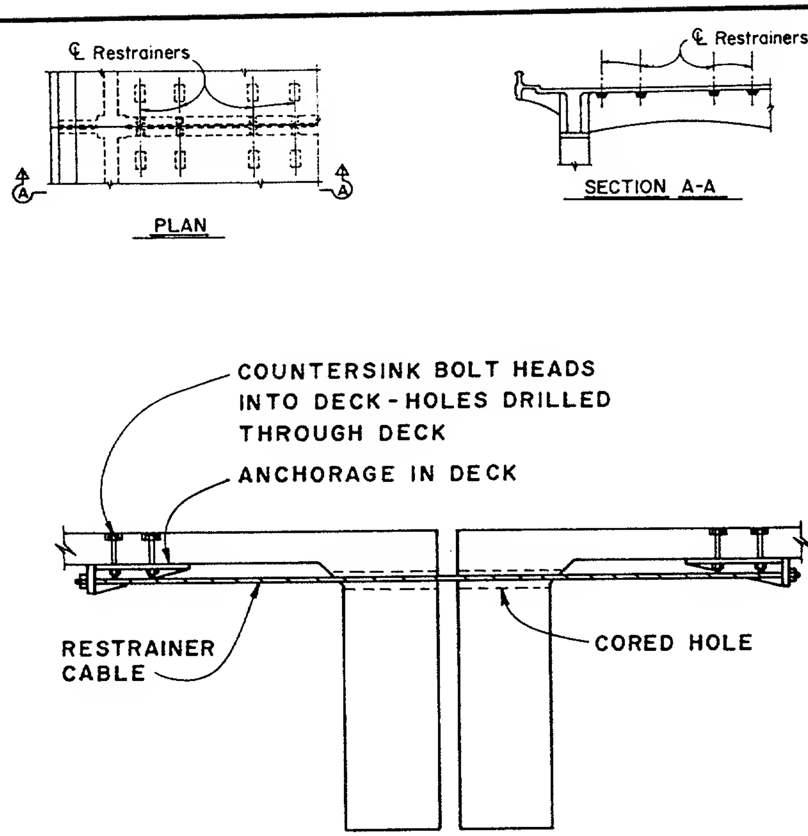


Figure 95: Expansion Joint Restrainers Tied to the Concrete Deck (from Reference 5)

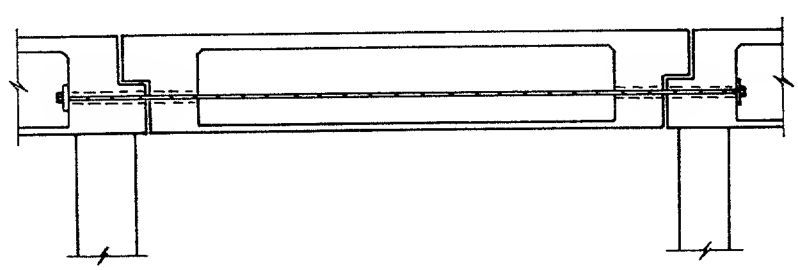


Figure 96: Restrainer Retrofit of a Suspended Span (from Reference 5)

One method that has been used to provide transverse restraint in concrete structures employs a double, extra strong, steel pipe filled with concrete that passes through the joint. This concept is shown in figure 97. The design is governed by bearing of the pipe against the walls of the cored hole. The full concrete compressive strength may be assumed in well-reinforced expansion joint diaphragms. Care should be taken not to use the full strength at acute corners in highly skewed joints because they can be very fragile.

Other devices, such as doweled concrete blocks, brackets bolted to supports, and transverse cables, can be used to solve unusual problems.

9.5.1C Vertical Motion Restrainers

The need for vertical motion restrainers will seldom be demonstrated by an analysis since vertical motions are not considered explicitly in the analysis requirements. However, experience has shown that vertical movement can take place at the bearings. This can lead to the displacement of bearings and possibly increase the chances of a loss of support failure. The Guidelines [reference 5] recommend that vertical restrainers be installed whenever longitudinal restrainers are considered as a retrofit measure and whenever the seismic uplift force obtained from an analysis of longitudinal motion exceeds fifty percent of the dead load reaction.

Vertical restrainers can be provided at a very low unit cost if they are installed at the same time as the longitudinal restrainers. A typical hold down detail is shown in figure 98.

9.5.1D Bearing Seat Extension

Bearing seat extensions may be a feasible retrofit measure in certain situations. Extensions allow larger relative displacements to occur at the joints before support is lost and the span collapses. Since high forces may be imposed on these extensions, it is recommended that, wherever feasible, they be supported directly on a foundation structure (such as at an abutment - figure 99). A bearing seat extension anchored with dowels or anchor bolts to a vertical face of an existing concrete support is not recommended, but if direct support on a foundation is impractical, post-tensioning of the extension should be considered.

The design forces for bearing seat extensions must be high to represent the large forces a bearing seat will be subjected to when the bearings become unseated. Two loading conditions are recommended. The first requires the extension to be designed to resist twice the vertical dead load reaction plus the maximum live load reaction. This is intended to account for the large impact forces that can result when the superstructure drops from the bearings onto the seat. The second requires the extension to be designed to resist a vertical load equal to the dead load reaction in conjunction with a horizontal load equal to the dead load reaction times either the acceleration coefficient or the friction coefficient between the two surfaces, whichever is the larger. This accounts for both the horizontal and vertical loads that can develop when the superstructure is resting on the bearing seat extension and still subjected to earthquake ground motions. Bottom surfaces of structures shall be modified to eliminate offsets due to bearing plates and the like, to avoid horizontal impacts against built-up seats.

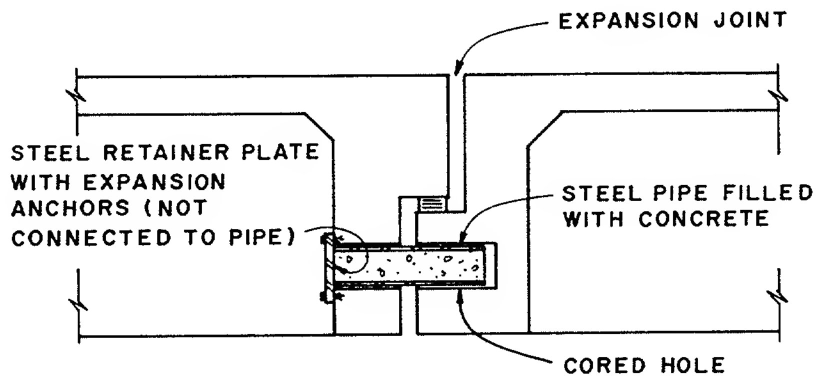


Figure 97: Transverse Restrainer Retrofit for Concrete Bridge (from Reference 5)

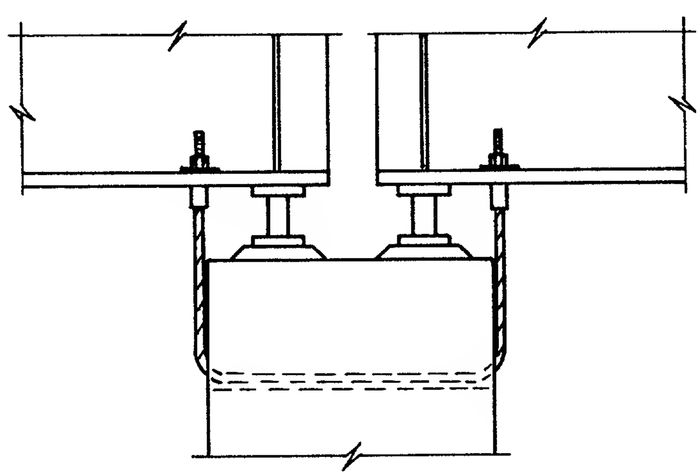


Figure 98: Vertical Motion Restrainer Retrofit (from Reference 5)

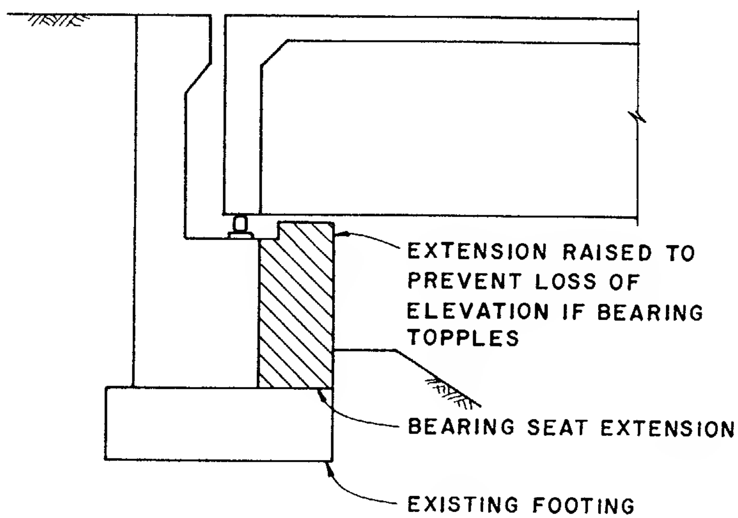


Figure 99: Bearing Seat Extension at Abutment (from Reference 5)

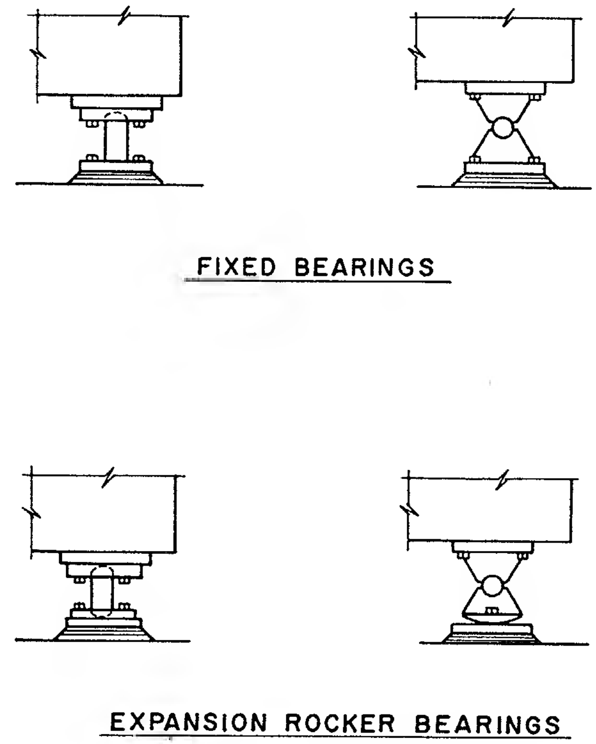


Figure 100: Seismically Vulnerable Bearings (from Reference 5)

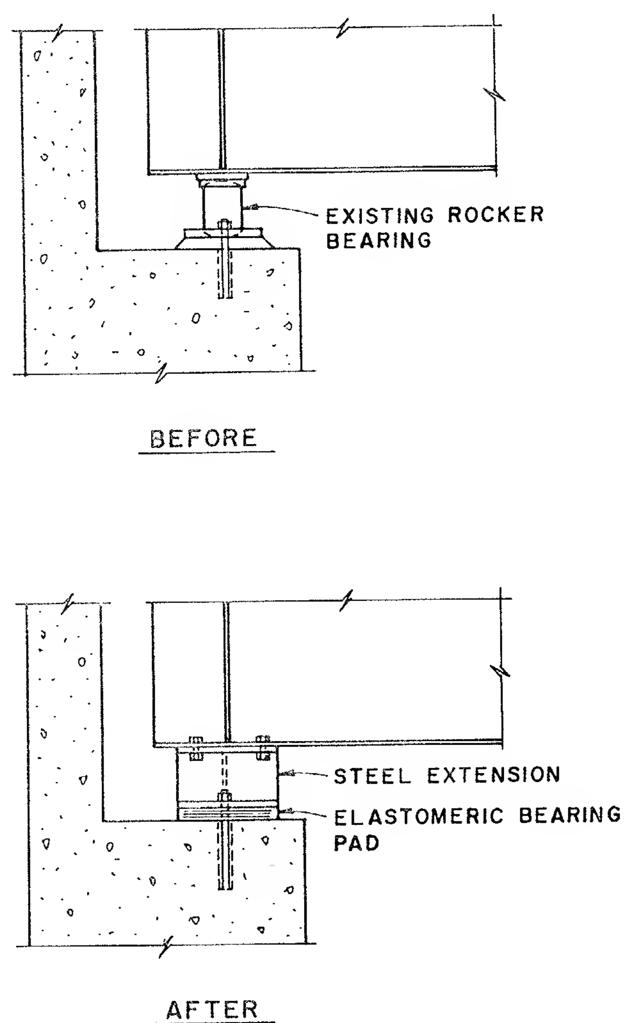


Figure 101: Replacement of Rocker Bearings – Steel Extension (from Reference 5)

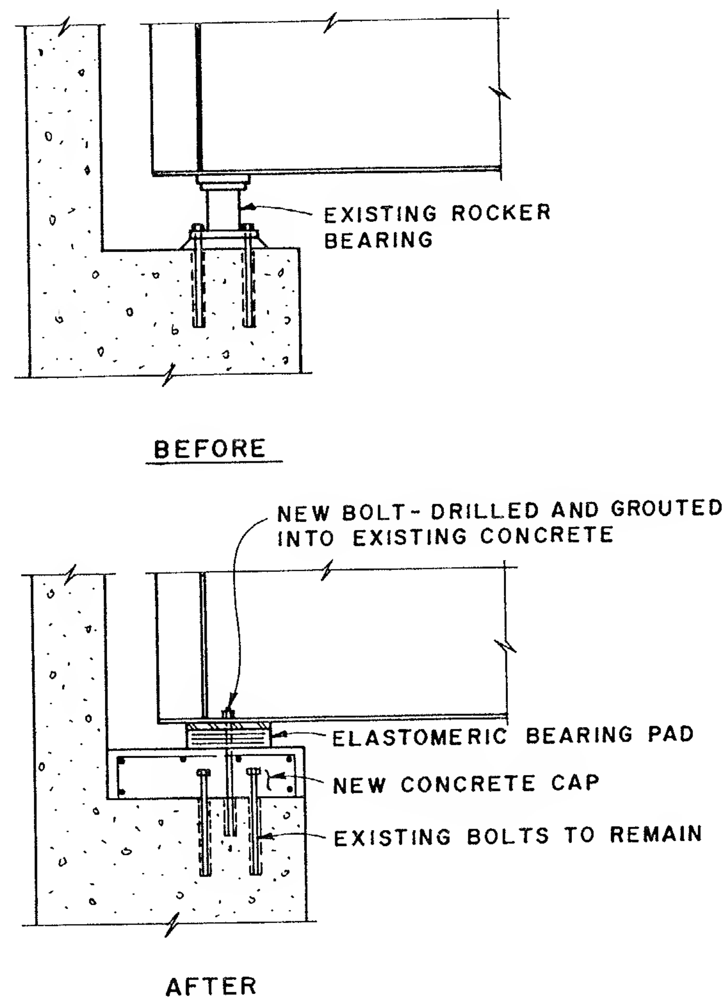


Figure 102: Replacement of Rocker Bearings – Concrete Cap (from Reference 5)

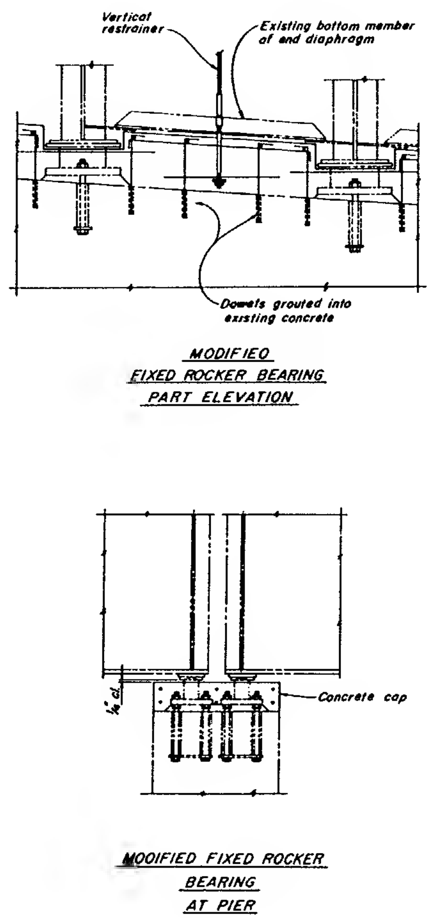


Figure 103: "Fixed" Bearing Retrofit by Embedment in Concrete (from Reference 5)

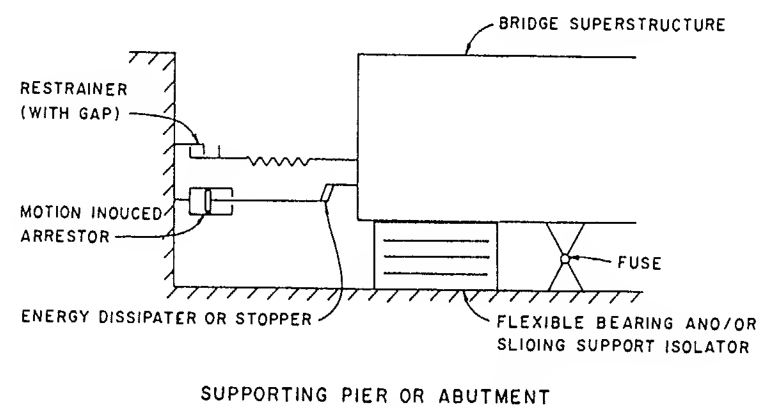


Figure 104: An "Ideal" Bearing Device (from Reference 5)

All bearing seat extensions should provide a final minimum seat width equal to or greater than the minimum specified value given in section 7.4.2 and 7.6.8.

9.5.1E Replacement of Bearings

Bearings which are damaged or malfunctioning can fail during an earthquake. In addition, certain types of bearings, such as steel rocker and roller bearings shown in figure 100, have performed poorly during past earthquakes. A possible retrofit measure in these cases is the replacement of the bearings with modern bearing types such as elastomeric pads or more sophisticated energy dissipating devices which, in conjunction with adequately designed restrainers, are more effective in resisting seismic loads.

Caltrans has used several methods of bearing replacement. In one method, high rocker bearings are replaced by a prefabricated steel bearing assembly and elastomeric bearing pads. The steel bearing assembly was necessary to maintain the proper elevation of the superstructure and to provide for the rotational and translational movement at the bearing. The details for this retrofit scheme are shown in figure 101.

Another possible solution to replacing steel rocker bearings is shown in figure 102. In this case a concrete cap is used to build up the elevation difference between a replacement elastomeric bearing and the original steel rocker bearing. With this method of replacement, the concrete cap can be constructed at a higher elevation between girders to provide a transverse shear key. In addition, vertical motion restrainers may be anchored in the new concrete cap.

At fixed bearings it may be appropriate to completely embed existing rocker bearing pedestals in concrete as shown in figure 103. This will prevent shear failure and toppling of the bearings. In addition, if spans were to become displaced from the bearings, the concrete cap would prevent collapse. Again the concrete cap can double as a shear key and anchorage for vertical motion restrainers.

Another recent application has been the replacement of existing bearings with more sophisticated energy dissipating devices. In addition to replacing vulnerable bearings, these dissipators also limit the seismic forces transmitted to the substructures. This is the basis of seismic isolation, and the concept is explained further in sections 4.7, 9.5.1F and 9.5.2.

9.5.1F Special Earthquake Resistant Bearings and Devices

Certain types of bearings have special performance characteristics which will alter the dynamic response of a bridge. As a consequence, superstructure forces can be reduced by factors of 5 to 10 and there are corresponding reductions in the forces transferred to the piers and abutments. Thus in addition to providing an acceptable replacement device for vulnerable bearings, they also provide a retrofit measure for understrength substructures, as discussed in section 9.5.2. Furthermore, selection of elastomeric bearings of different height and shear stiffness can be used to control the distribution of lateral load as discussed previously in section 5.1.4.

More rigorous analysis procedures should be used when bearings and devices of this kind are used. This is particularly important if they use nonlinear characteristics to achieve the desired force reductions. If a nonlinear time-history analysis is performed,

at least three ground motion time histories should be used. These ground motions should have different frequency content and duration of maximum shaking. They should also reflect the variations in ground motions expected at the bridge site. If design charts have been developed they may be used in lieu of a nonlinear analysis, provided they are based on a series of nonlinear analyses consistent with the above-stated criteria.

In addition to performing under normal service conditions, an earthquake resistant bearing should be capable of resisting seismically induced forces, restricting relative displacements within the bridge, dissipating energy, and returning the structure to its pre-earthquake position. A bearing system having these capabilities has been discussed in section 4.7 and might be composed of the components shown in figure 104. Vertical support would be provided by a flexible bearing and/or sliding support isolator. In the case of a flexible bearing, a fuse would be used to prevent movement under service conditions, but would be expected to fail or yield during a large earthquake. During rapid movement, energy would be dissipated by some form of damper, and excessive relative displacements would be prevented by a restrainer with a gap to allow limited displacements. Following an earthquake, the flexible support would provide a restoring force to bring the structure back to its pre-earthquake position.

A considerable amount of interest currently exists in the improvement of bearing systems to provide greater earthquake resistance. In New Zealand, Italy, Japan and, more recently, the United States, many innovative ideas have been implemented in this field. These ideas use the principles of restraint, isolation, and energy dissipation to modify structural behavior during earthquakes. In each case these bearing systems also provide for the normal functions of bridge bearings. A few examples are discussed below.

New Zealand has constructed 37 bridges using special energy-dissipating devices. Some of the early devices are shown in figure 105. All of these devices rely on the inelastic behavior and hysteretic damping that will occur during reversed cycles of yielding. The devices shown are used to connect the bridge superstructure to the substructure and are usually installed in parallel with elastomeric bearing pads. At low levels of lateral load such as wind, the devices remain elastic and restrain movement at the bearings. During strong seismic shaking, the devices yield, allowing translation at the bearings. When they yield, the load transmitted from the superstructure to the substructure will be limited to the ultimate capacity of the devices. In addition, energy will be dissipated during yielding which will dampen the seismic response.

One refinement has been to combine the energy dissipator with the elastomeric bearing in one physical unit. To do this, a circular core is removed from an elastomeric bearing and the hole backfilled with lead, as shown in figure 106. During cyclic shear deformations of the bearing, the lead core is forced to deform in shear also. Plastic shear deformations in the lead dissipate significant amounts of energy and thereby limit the shear displacements in the bearing. This unit is popular in New Zealand and has now been used, or proposed for use, in 25 bridges.

Because lead has a low rate of work hardening, it can sustain many cycles of imposed deformation due to thermal and creep movements without fracture. In addition, its resistance to slowly applied deformation is less than half of that which will occur under rapid movement. This makes it possible to use the device as an expansion bearing. Under rapid movements, which occur during a strong earthquake, the lead will resist greater loads and dissipate energy. In addition there will be a reduction in the total

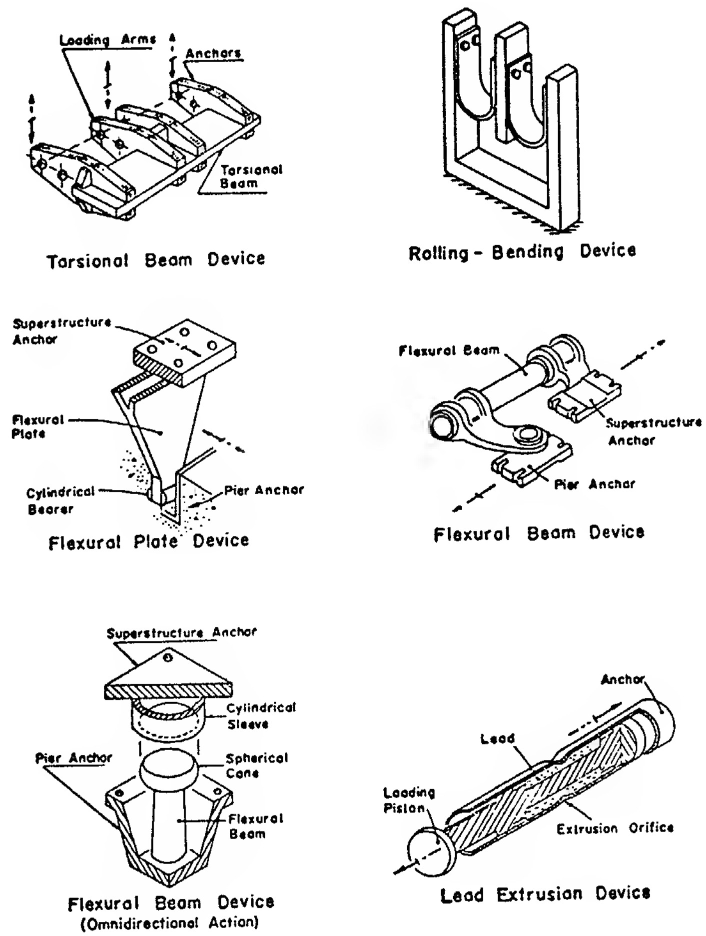


Figure 105: New Zealand Energy Dissipation Devices (from Reference 5)

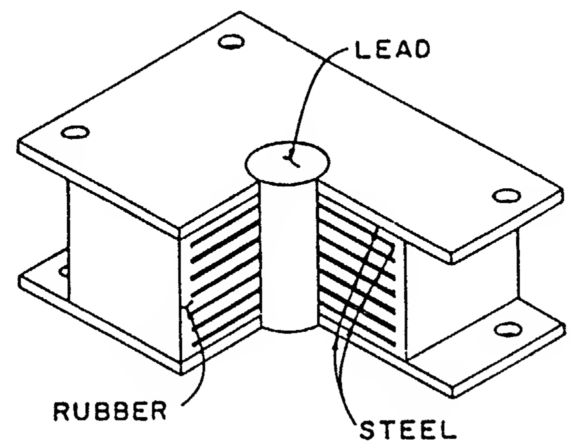


Figure 106: Elastomeric Bearing Pad With a Lead Core (from Reference 5)

earthquake load due to energy dissipation. Bridge retrofit using lead-filled bearings has been undertaken in New Zealand, Italy and now in the United States. These bearings are patented in New Zealand, Japan, the United States and elsewhere.

In Japan a somewhat similar philosophy for bridge bearings has been adopted. The Japanese, however, use viscous rather than hysteretic damping to achieve this type of performance. Load transfer to the substructure occurs when a rigid post is forced through a pot of viscous material as shown in figure 107. Since thermal movements and creep occur very slowly, little resistance is offered and negligible loads are transferred under these conditions. However, during an earthquake the rapid movement of the post is resisted by the viscous material, and significant load is then transferred to the substructure. A combination bearing and shear damper has also been developed (figure 108) and used extensively. It is also a patented product.

A viscous damping device is used on the new Dumbarton Bridge across the southern end of San Francisco Bay in California. This device, shown in figure 109, allows the expansion joint to open and close during normal temperature movement but limits the relative movement of the joint during an earthquake.

Some oil damper systems are known to leak and all require regular maintenance and inspection. Because their reliability is low, a positive back up system such as an elastic restrainer is recommended to prevent catastrophic failure.

An expansion bearing design concept, developed and tested during a study for the Federal Highway Administration, employs an elastomeric bearing pad surfaced with a special material designed to slide during seismic loading. Under normal conditions the bearing will perform as a standard elastomeric bearing pad. At higher loads, sliding will occur and limit the load transferred to the supports, protect the pad from being destroyed, and maintain the reliability of vertical support. This bearing concept, shown in figure 110, has the disadvantage that it will be permanently offset after an earthquake.

9.5.2 Columns, Piers and Footings

There are a number of potential modes of failure for columns, piers and footings and in general, it is more difficult and less cost effective to retrofit these components than it is to upgrade the bearings. Very few column retrofit techniques have been used in practice and it appears that the only practical, cost-effective method currently available is the use of the force limiting devices and seismic isolation bearings discussed earlier.

Force limiting devices and some of the other schemes that have been proposed are presented below. Concepts that have not been used are also presented primarily as a source of ideas.

9.5.2A Force Limiting Devices

A force limiting device which uses the principles of seismic isolation (section 4.7) is the most practical and cost-effective method currently developed for column retrofit. These devices have been discussed in detail in section 9.5.1F and when used in conjunction with a bearing have the potential to reduce the real forces, to which a column is subjected, by factors of 5 to 10. Thus the demand is significantly reduced and the C/D ratios are significantly improved.

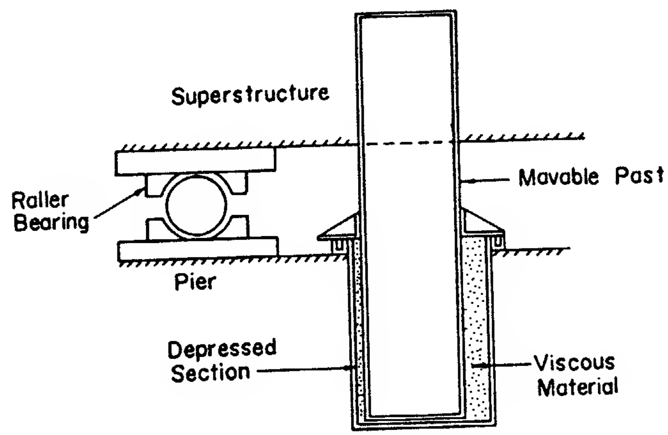


Figure 107: Japanese "Earthquake Stopper" Device Used on the Ohtagawa Railway Bridge (from Reference 5)

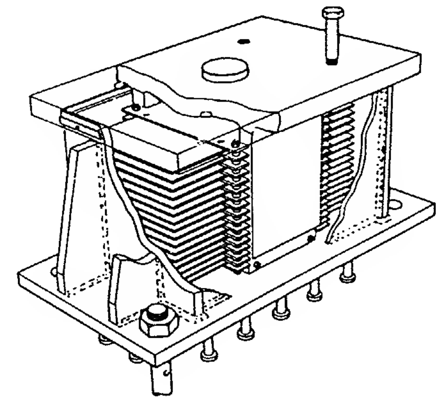


Figure 108: Viscous Shearing Type Damper (from Reference 5)

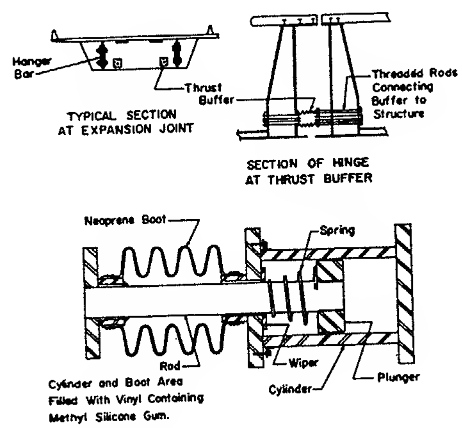


Figure 109: Viscous Damping Devices Used on the New Dumbarton Bridge (from Reference 5)

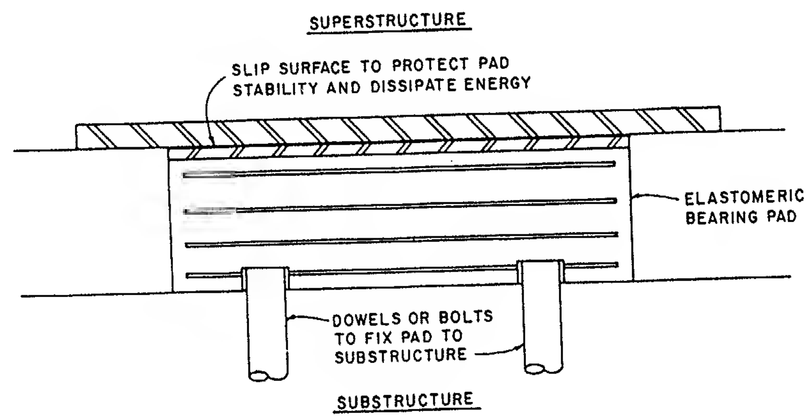


Figure 110: Sliding Elastomeric Bearing Pad (from Reference 5)

The use of force-limiting devices should be restricted to devices whose dynamic performance has been demonstrated by physical testing. Design forces and displacements should be derived from an analysis of the structure which takes into consideration the actual performance characteristics of the device.

9.5.2B Increased Transverse Confinement

Improved confinement will increase the ability of a column to withstand repeated cycles of loading beyond the elastic limit and tend to prevent column failure due to degradation of flexural capacity. The use of the detailed design requirements for transverse reinforcement for new bridges in a retrofit situation will present construction difficulties and will be of questionable effectiveness.

Several different concepts have been proposed but so far as is known, none have been used to date. The following design requirements should be considered when evaluating these ideas.

Increased transverse confinement should be located within the column end regions. The end regions should be assumed to extend from the soffit of the girders or cap beams at the top of columns, or the top of foundations at the bottom of columns, a distance not less than the greater of (a) the maximum cross-sectional dimension of the column, (b) one-sixth of the clear height of the column, or (c) 18 inches.

The transverse confinement should be capable of developing the confining force provided by the transverse confinement required for new construction. In addition, if the capacity/demand ratio for shear in the existing column is less than 1.0, the transverse reinforcement should be capable of resisting the maximum shear force due to hinging in the column. It should also be remembered that the transverse reinforcement must extend over the full column height if shear and not flexural confinement controls. Transverse confinement reinforcing should have a maximum spacing not to exceed the smaller of one-quarter of the minimum member dimension or 4 inches. It is noted that current Caltrans requirements relax the maximum spacing to the smaller of 8 main column bar diameters or 8 inches. Anchorage schemes for transverse reinforcement should be capable of developing the ultimate capacity of the reinforcement, and should not be significantly affected by the spalling of cover concrete. The designer should be aware that retrofit schemes for increasing confinement may redistribute moments and shears, resulting in overstress in other members of the structure, i.e., footings and bent caps. Increased transverse confinement should result in capacity/demand ratios greater than one for each of the column failure modes. If this is not the case, then additional retrofit measures should be considered.

Several methods of increasing the transverse confinement of columns have been proposed.

One proposal which uses conventional half-inch steel reinforcing hoops, prestressed onto the outer face of the column is shown in figure 111. The prestress force is provided by threading the ends of the bars so that these can be connected together with a specially designed turnbuckle, also shown in figure 111. The steel bars would be spaced at 3-1/2 inches on center which would provide confinement equivalent to new construction in most cases. The steel would be protected with a layer of pneumatically applied concrete.

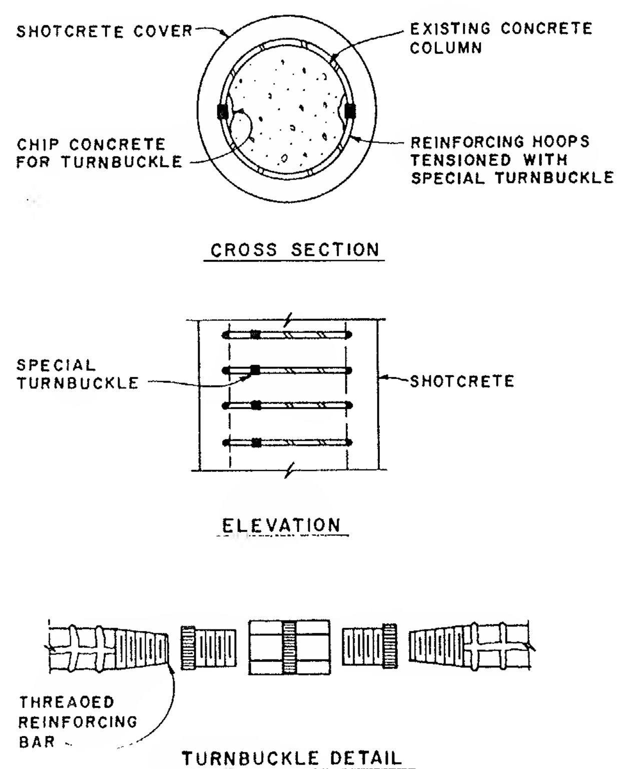


Figure 111: Column Retrofit to Increase Confinement with Steel Hoops (from Reference 5)

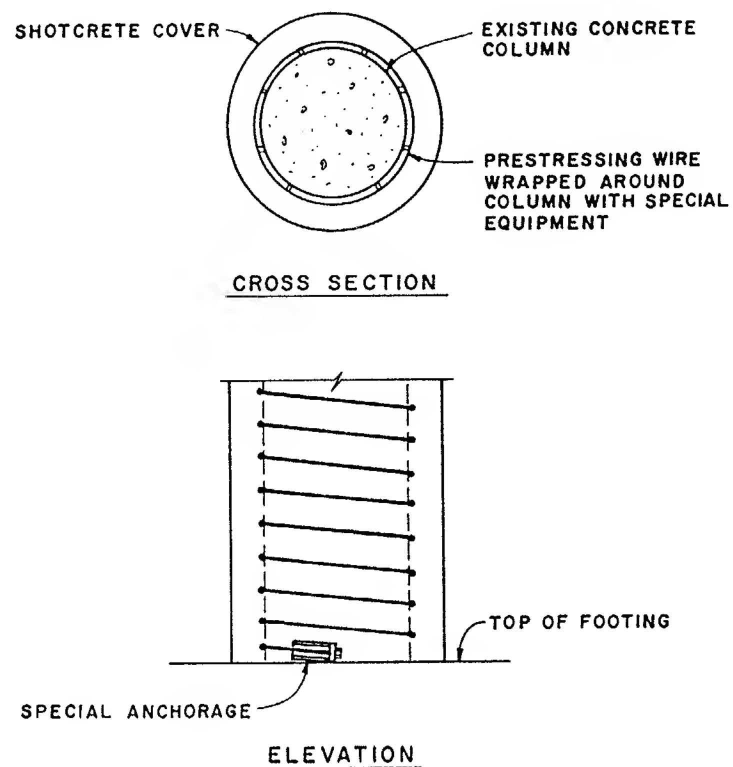


Figure 112: Column Retrofit to Increase Confinement with Prestress Wire (from Reference 5)

Another proposal to use quarter-inch prestressing wire wrapped under tension around the column is shown in figure 112. The wire and anchorages would also be protected with a layer of pneumatically applied concrete. The practical difficulties associated with wrapping a very long piece of prestressing wire around a bridge column should not be underestimated.

A solid-steel shell placed around an existing column, as shown in figure 113, has also been proposed. A small space would be left between the column and the shell that would be grouted solid. The steel shell could be painted or it could be constructed of a weathering type of steel.

Square columns pose additional problems. Aesthetics and clearances are other considerations which will dictate solutions.

9.5.2C Reduced Flexural Reinforcement

The ultimate shear force on a column can be reduced by decreasing the yield moment at one or both ends of the column. This retrofit method should only be considered when columns are over-reinforced for flexure and when there is little or no flexural yielding during an earthquake. The high-yield moments of an over-reinforced column could produce shear forces above the capacity of the column resulting in a brittle shear mode of failure. By cutting longitudinal reinforcing bars (figure 114), an increased amount of yielding is accepted in exchange for a reduced shear force. The net result could be an improvement in the overall earthquake resistance of the structure. Despite its conceptual appeal, it is controversial because it does reduce the flexural strength of the column. It is recommended that cutting of column longitudinal reinforcement as a retrofit measure be used only when it is not possible to retrofit the column by other means.

9.5.2D Increased Flexural Reinforcement

The use of increased flexural reinforcement has also been proposed. This retrofit technique will increase the flexural capacity of the column. However, increased flexural capacity will increase the forces transferred to the foundation and the superstructure/column connections and will also result in an increased column shear force. In addition, the strengthened column will be stiffer and may be subjected to higher seismic forces. Since failure of the footings or failure of the columns in shear is usually more critical than excessive flexural yielding, this retrofit technique should be used with care and should only be considered when loss of flexural strength would result in a collapse mechanism.

Retrofit methods used by the Japanese to increase the flexural strength of reinforced concrete building columns are shown in figures 115, 116, and 117.

9.5.2E Infill Shear Wall

The transverse resistance of multi-column bents can be increased by constructing an infill concrete shear wall between individual columns in the bent. This technique has been used to repair earthquake damage to bridges in Japan and California, and requires that individual column footings be extended to support the shear wall. The shear wall is tied into the existing structure with grouted bars or anchors.

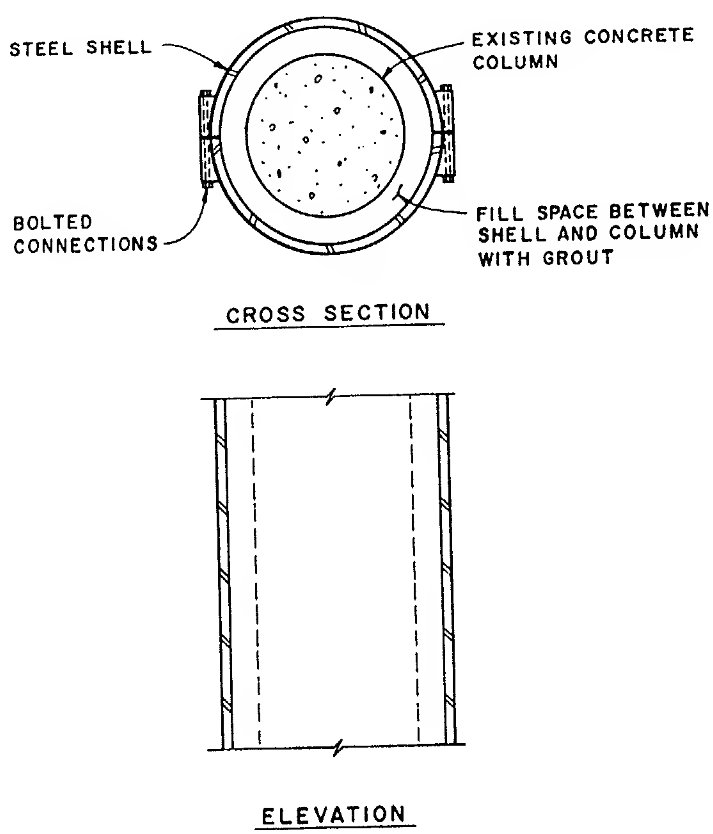


Figure 113: Column Retrofit to Increase Confinement with Steel Shell (from Reference 5)

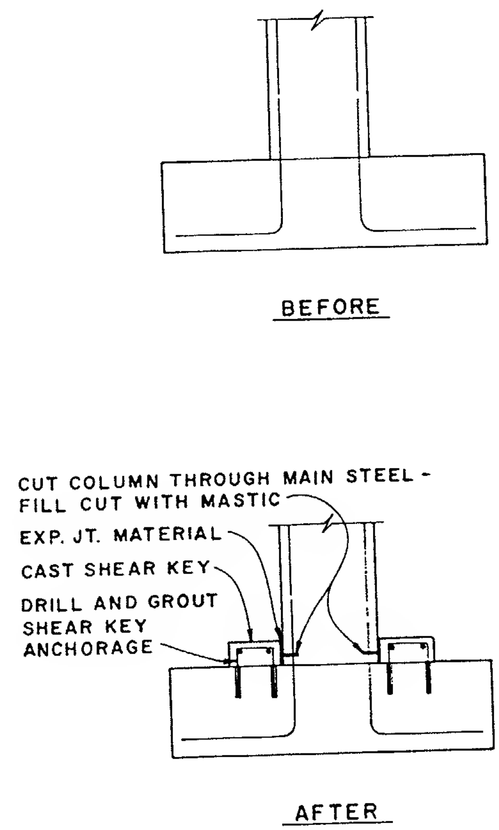


Figure 114: Column Retrofitting by Cutting Main Reinforcement (from Reference 5)

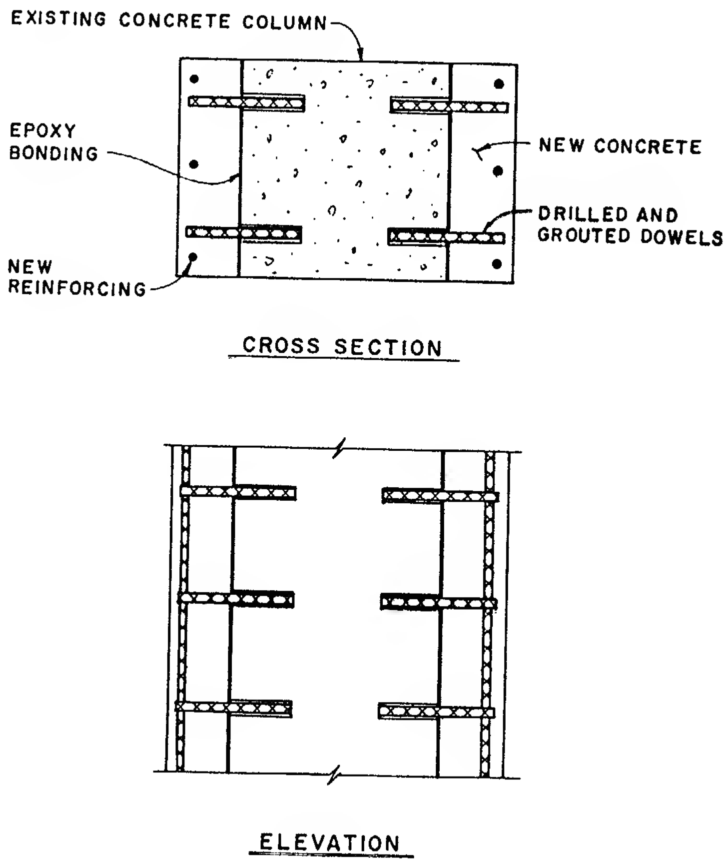


Figure 115: Column Strengthening by Concrete Overlays (from Reference 5)

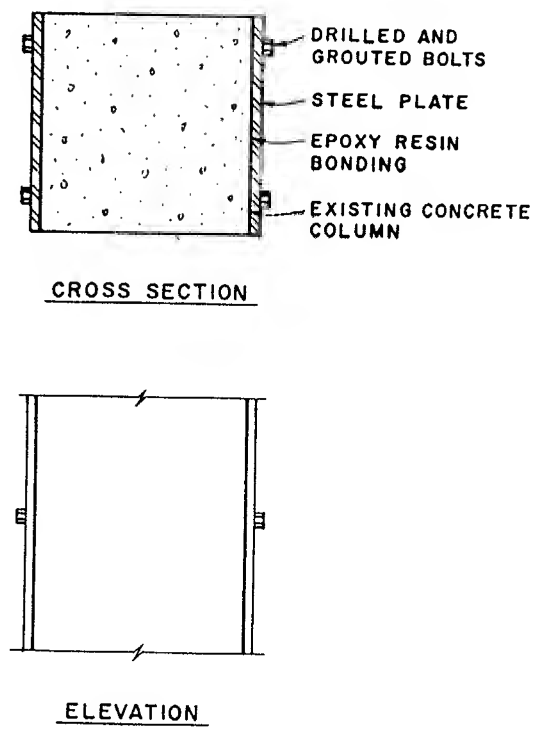


Figure 116: Column Strengthening by Steel Plates (from Reference 5)

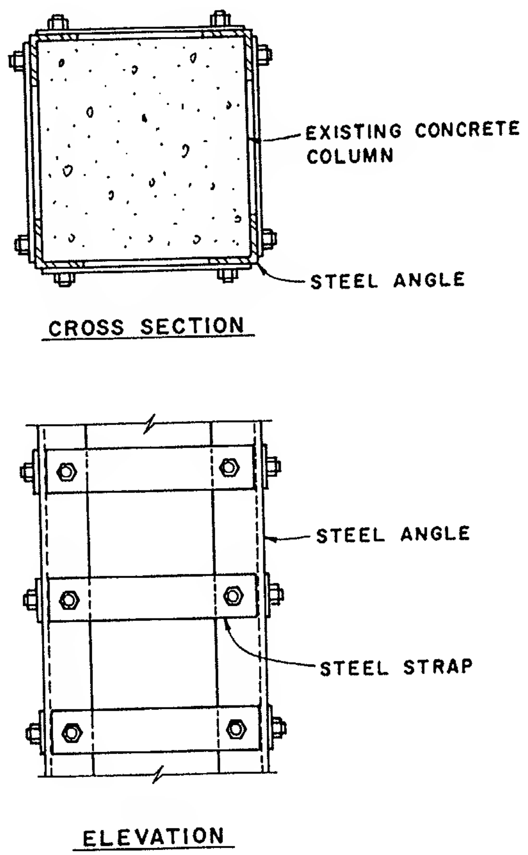


Figure 117: Column Strengthening by Steel Angles (from Reference 5)

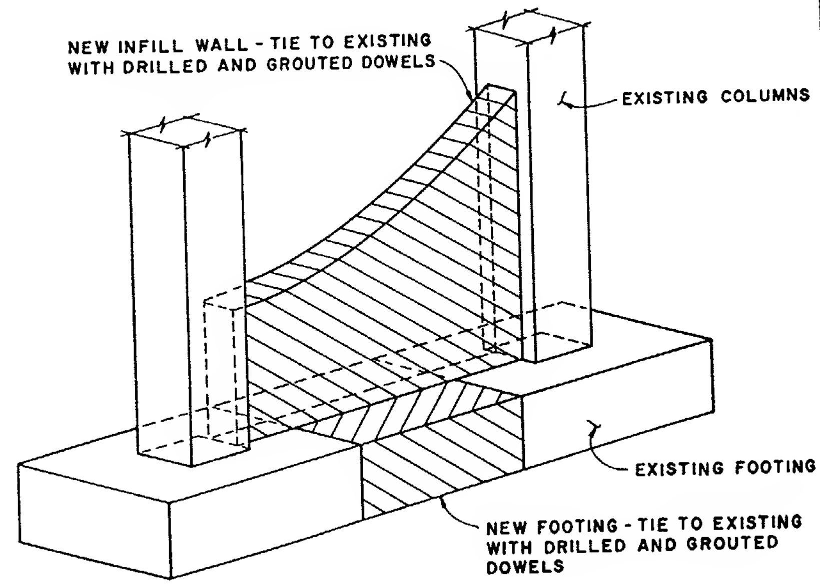


Figure 118: Infill Shear Wall Retrofit (from Reference 5)

Figure 118 illustrates the use of a infill concrete shear wall to retrofit a multi-column bridge bent. This type of structural modification will have a significant effect on the structural strength and stiffness in the transverse direction and will require a new analysis to be made in order to obtain the revised design forces.

9.5.2F Strengthening of Footings

In many cases, column footings will fail before the column or pier yields. This is often due to the absence of a top layer of reinforcement capable of resisting uplift forces on the footing. During an earthquake, this can result in the flexural cracking of footing concrete and the loss of anchorage for the column longitudinal reinforcement. This condition is usually most critical in single column piers supported on pile footings.

Although minor soil and piling failures are undesirable, these are preferable to the failure of the structural components of a footing. Therefore, in the case of retrofitting, footings should be strengthened so that they do not fail prior to soil or piling failure. Design moments and forces should therefore be equal to 1.25 times the nominal ultimate capacity of the soil and/or pilings.

A method for retrofitting footings to correct this deficiency is shown in figure 119. A concrete cap of constant thickness is cast directly on top of the footing. Continuity with the existing footing is provided by steel dowels grouted in drilled holes. Negative moment capacity is provided by a top layer of conventional reinforcement and prestress tendons. The collar will strengthen the footing to resist uplift forces and provide an extra measure of confinement at the base of the column and the top of the footing to prevent anchorage failures.

9.6 ABUTMENTS

Abutment failure very rarely results in the collapse of the structure unless associated with liquefaction failure or loss of support for the end spans. For example, seat type abutments on high walls which are skewed to the bridge spans are particularly prone to loss-of-support failures. Lateral movements of an earth-retaining abutment or consolidation of the abutment fill may result in a loss of accessibility to the bridge, which may be an unacceptable failure for an important bridge. In addition, the use of restrainers to limit relative displacements at the abutment bearings may result in much larger abutment forces. Therefore, situations will exist in which abutment retrofitting should be considered. The following paragraphs discuss two possible retrofit measures that will mitigate the effects of abutment failure.

9.6.1 Settlement Slabs

Settlement (or approach) slabs are designed to provide continuity between the bridge deck and the abutment fill in the case of approach fill settlement. Settlement slabs should be positively tied to the abutment to prevent them from pulling away and becoming ineffective. It is recommended that they be considered only for bridges classified as SPC-D with approach fills subject to excessive settlement due to either soil failure or excessive movement of the abutment. To minimize the discontinuity at the abutment following an earthquake, settlement slabs should be designed as simple span-reinforced concrete slabs spanning their full length. Positive ties to the abutment

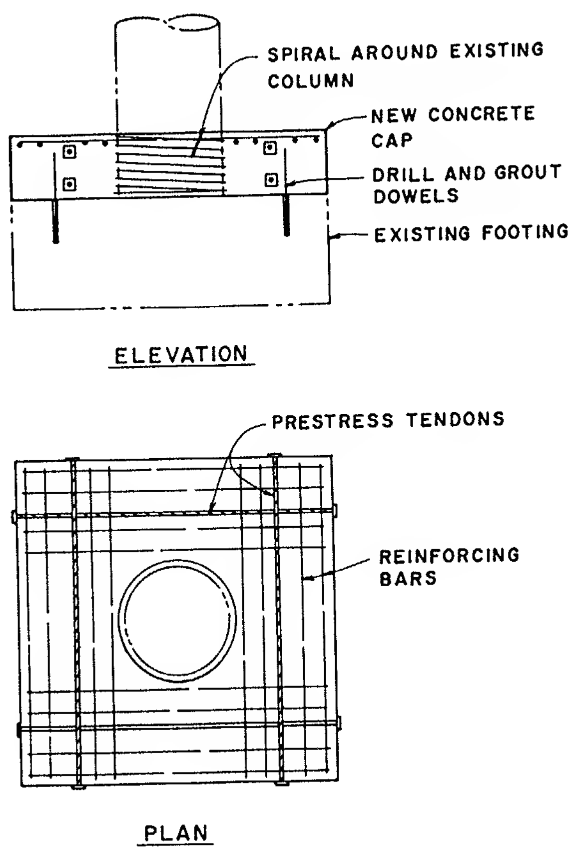


Figure 119: Footing Retrofit by Adding Concrete Cap (from Reference 5)

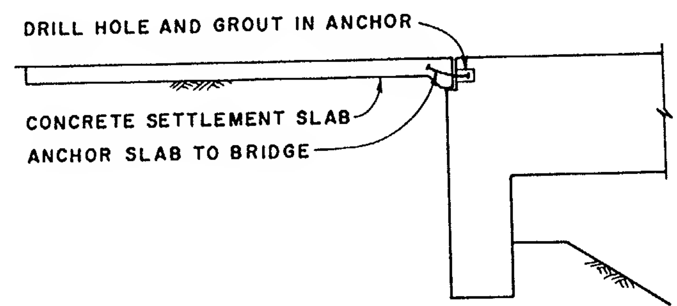


Figure 120: Settlement Slab - California Style (from Reference 5)

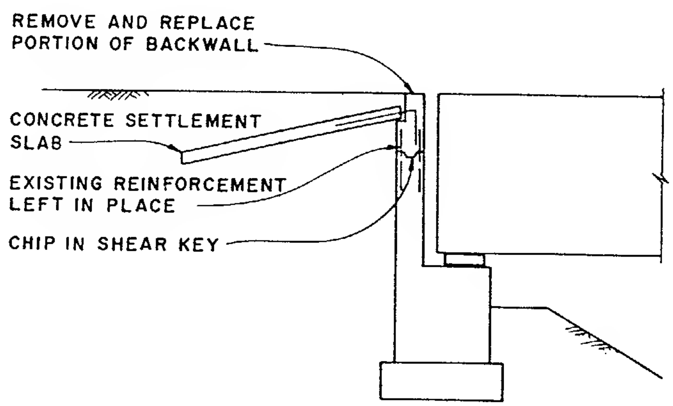


Figure 121: Settlement Slab – New Zealand Style (from Reference 5)

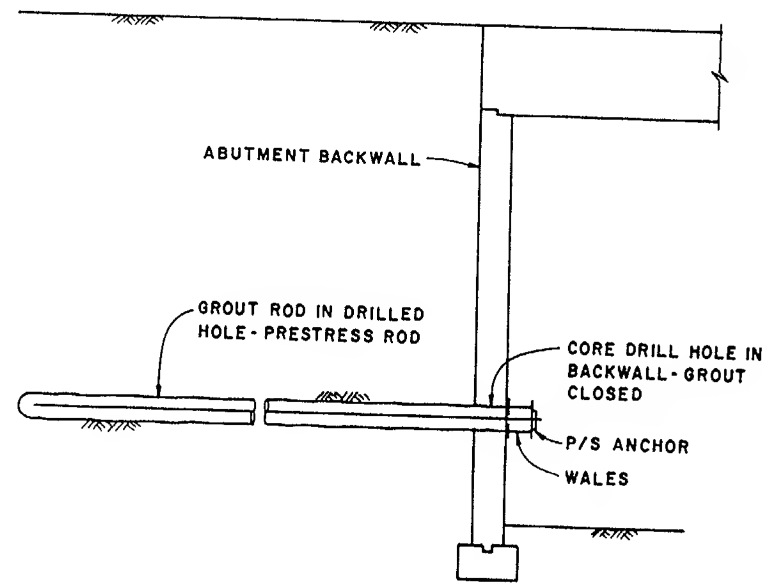


Figure 122: Retrofit of Abutment with Soil Anchors (from Reference 5)

should be capable of resisting a design force equal to (coefficient of friction + Acceleration Coefficient) x slab dead load.

It should be pointed out that this connection should be free to rotate so that moment will not be transferred to the abutment backwall when the approach fill settles.

Figures 120 and 121 show two different types of settlement slabs that have been used in the past. In figure 121, the frictional force due to the weight of the soil above the slab may help stabilize the abutment. However, this effect is not expected to be large and both configurations should perform in a similar manner.

9.6.2 Soil Anchors

Horizontal displacements at the abutment may cause a loss of accessibility to the bridge. Displacements of the abutment normal or parallel to the abutment face may be prevented or minimized by adding soil anchors.

The ultimate capacity of soil anchors should be greater than or equal to the seismic forces transferred to the abutment from the superstrucure and/or the seismic earth pressures generated behind the abutment backwall due to the design earthquake.

Soil anchors similar to those shown in figure 122, may be used as a retrofit measure. Because the backfill may be subject to movement during an earthquake, the anchors should extend into the backfill a sufficient distance so as not to be affected.

9.7 LIQUEFACTION AND SOIL MOVEMENT

Liquefaction and/or excessive movement have been the cause of a large number of bridge failures in some areas during past earthquakes. There are two suggested approaches to retrofitting that will mitigate these types of failure. The first approach is to eliminate or improve the soil conditions that tend to be responsible for seismic liquefaction. The second approach is to increase the ability of the structure to withstand large relative displacements similar to those caused by liquefaction or large soil movement. The first approach has been tried on dams, power plants, and other structures but to date has not been used as a retrofit measure for bridges. The second approach utilizes many of the retrofitting techniques in the previous sections. Both of these approaches are outlined in the following sections. Assessment of the potential for site liquefaction is discussed in references 4 and 6.

9.7.1 Site Stabilization

Although site stabilization would only be used in exceptional cases, several methods are available for stabilizing the soil at the site of the bridge. Some possible methods include:

- Lowering of groundwater table.
- Consolidation of soil by vibroflotation or sand compaction.
- Vertical network of drains.
- Placement of permeable overburden.
- Soil grouting or chemical injection.

Some of these methods may not be suitable or environmentally acceptable, and may even be detrimental in certain cases unless provisions are made to minimize the effects of soil settlement during construction. Therefore, careful planning and design are necessary before employing any of the above site-stabilization methods. Each method should be individually designed using established principles of soil mechanics to ensure that the design is effective and that construction procedures will not damage the existing bridge.

The first method suggested is to lower the ground water table. This eliminates the presence of water, which is one of the three items required before liquefaction can occur. The possibility and expense of accomplishing this will depend on the site. Obviously, some type of gravity drainage would be preferred to mechanical methods although mechanical methods such as well points are not out of the question in a major structure of unusual importance. Drainage can cause settlement of the surrounding soil and the effect of this settlement on the existing bridge should be assessed before this method is used.

Densification of the soil can also be effective in reducing the potential for liquefaction. Since the process of liquefaction involves the compaction of loose soil, it follows that preconsolidation can reduce the risk of liquefaction. However, consolidation of only the surface layer can impede drainage and actually be detrimental. Soil densification through the use of vibroflotation or sand compaction piles improves drainage if porous material is used and therefore is the preferred method. Preconsolidation can result in significant settlements, and care should be taken to protect the existing structure from damage. Often excessive settlements during construction will make soil densification an impractical retrofit method.

A method which will improve drainage without disrupting the existing structure is to install a network of gravel drains as shown in figure 123. These drains will allow water to escape during an earthquake and thus prevent the build-up of pore pressure which can reduce the shear strength of the soil. Settlement will be likely during an earthquake, but large lateral movements resulting from shear strength loss will be greatly reduced.

The use of a highly porous overburden or surcharge can also greatly reduce liquefaction potential with minimal disruption to the existing structure. The increased intergranular forces resulting from the overburden will necessitate higher pore pressures to offset these forces and cause liquefaction. The permeability of the overburden will not aggravate the build up of pore pressure. In addition, the overburden will result in some preconsolidation which will reduce the chances of liquefaction. However, the settlements that will accompany this preconsolidation should be considered when using this approach.

The use of chemicals or grouts to increase the shear strength of soil is also a possible solution. If not properly designed, these methods may reduce soil permeability and aggravate the build-up of pore pressure. Therefore, design and construction should be performed by qualified individuals.

9.7.2 Increased Superstructure Continuity and Substructure Ductility

Any method that will tend to prevent loss of support at the bearings will be useful in preventing structural collapse due to excessive soil movement. Therefore, most of

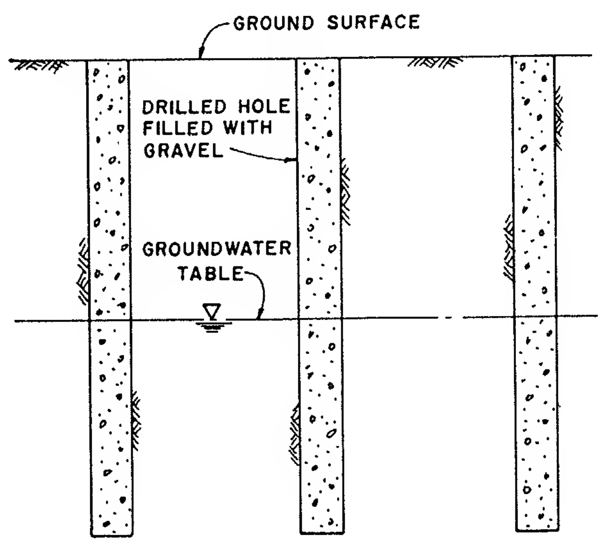


Figure 123: Gravel Drain System (from Reference 5)

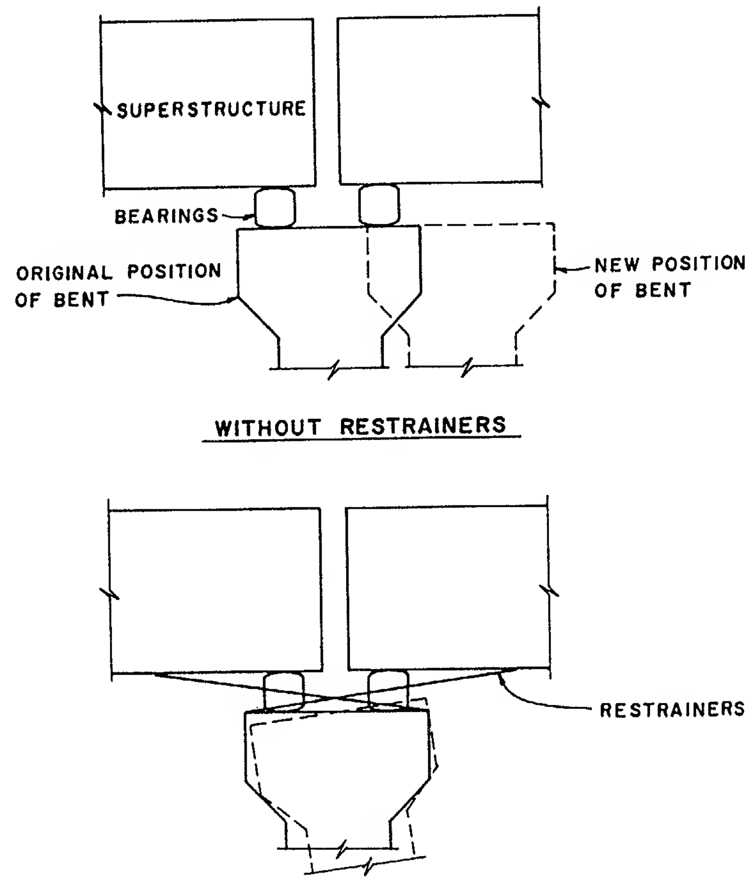


Figure 124: Effects of Restrainers at Bent During Liquefaction Failure (from Reference 5)

the methods for retrofitting bearings should be considered in a structure subjected to excessive soil movement. In addition, the ability of the substructure to absorb differential movements is important. The strengthening methods used will depend on the configuration of the structure and components most susceptible to damage. These will usually involve methods for tying superstructure sections together and connecting the superstructure to the bents. In some cases, column retrofitting should be considered. Attempts to stabilize the abutments through the use of anchors would probably not be very effective.

Longitudinal restrainers should be provided at the bearings to prevent a loss of support. If bents are not tied to the superstructure, the movements of the foundation can easily pull the support out from under the bearings as shown in figure 124. It would be preferrable to fail the column in flexure rather than to lose this support. Therefore, the superstructure should be anchored to the bent, and the design load in the anchors should be at least enough to fail the bent. Care should be taken to provide a sufficient gap in the restrainers so that normal temperature movement or moderate earthquakes will not result in a column failure. Gaps should be preset so that restrainers just fit snugly in cold weather.

Transverse and vertical restrainers at the expansion joints tend to prevent the superstructure from shifting and should be used along with longitudinal restrainers. When expansion joints occur at the bents, these restrainers should provide a positive tie to the substructure.

Because ductile failures of the columns are required to accommodate large movements, column retrofitting may be necessary to assure that a brittle failure does not occur. Extra transverse reinforcement or reduction of flexural capacity are two possible retrofitting techniques for accomplishing this.

CHAPTER 10 COMPARATIVE ANALYSES

In this chapter, several comparative analyses are reported for two different bridges. Both are continuous over several spans, but one is straight and the other is curved.

For the straight bridge, several variations are analyzed in which pier heights are changed, skew is introduced and abutment restraint is relaxed. Results from both the single mode and multi-mode methods of analysis illustrate the effect of these changes on the seismic response and show some limits of single mode modelling.

The computer program SEISAB was used throughout to perform these analyses and this chapter therefore also demonstrates the user-friendly nature of this program and its ability to analyze complex structures with ease.

10.1 STRAIGHT BRIDGE EXAMPLE

10.1.1 Geometry

A three span continuous girder bridge is supported on multicolumn bents as shown in figure 125. Properties for both the superstructure and substructure are given in this same figure. The abutments are assumed to be seat-type abutments which are free to slide in the longitudinal direction but are restrained transversely. It will be noted that this is the same bridge as that used in chapter 8 to illustrate the seismic design procedures.

In addition, this basic bridge geometry is modified seven times to produce another seven examples for analysis. These modifications are

- a change in column height for bent 3 from 25 feet to 50 feet.
- introduction of 45° skew at all abutments and piers.
- release of the transverse restraint at both abutments.
- combinations of the above.

Table 23 identifies each of these examples and the variation made to the basic bridge model. In total, the analysis of eight examples is described in this section.

10.1.2 Load Cases

The response spectrum chosen for all eight analyses was as follows:

Acceleration Coefficient, $A = 0.4$
Soil Profile Type: II, $(S = 1.2)$
Seismic load coefficient, $C_s = 1.2AS/T^{2/3}$
$= 0.576/T^{2/3}$

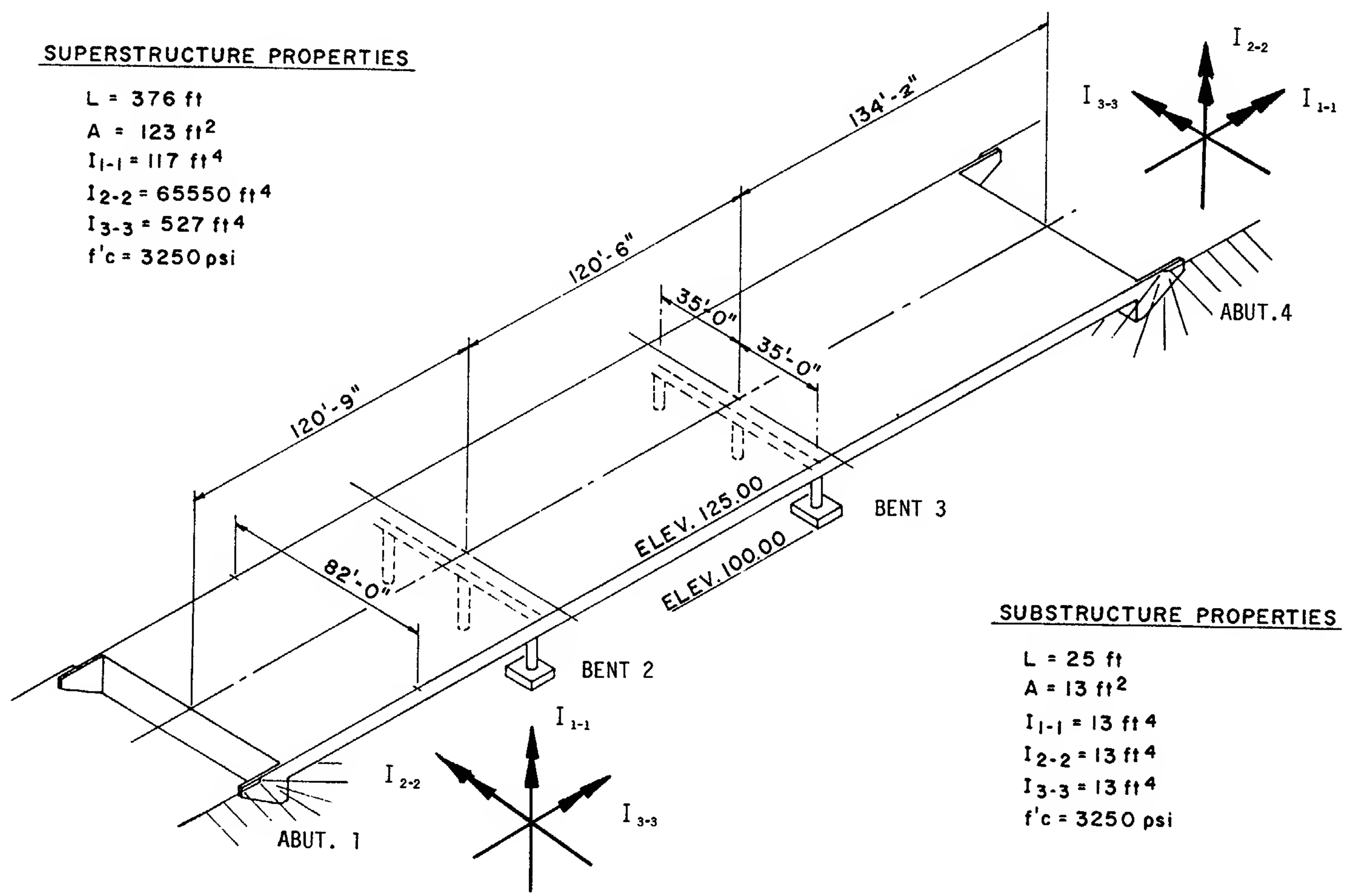


Figure 125: Three-Span, Straight Bridge Example

Table 23: Straight Bridge Example

EXAMPLE	DEFINITION
A	Base Example: 3-span straight bridge, all column heights 25 ft.
B	Same as A, but with column height equal to 50 ft. at bent 3
C	Same as A, but with abutments and bents skewed 45 degrees
D	Combine B + C (unequal columns and skew)
E	Same as A, but release both abutments transversely
F	Combine B + E (unequal columns and free abutments)
G	Combine C + E (skew and free abutments)
H	Combine B + C + E (unequal columns, skew, and free abutments)

Direction of loading: longitudinal and transverse

Load combinations were made as follows:

Case 1: 1.0 * longitudinal + 0.3 * transverse
Case 2: 0.3 * longitudinal + 1.0 * transverse

10.1.3 Analysis Methods

Both **single mode** spectral analysis (procedure 1) and **multimode** spectral analysis (procedure 2) were used to analyze each of the eight bridges in turn to determine the accuracy of the single mode method for this series of regular bridges of increasing complexity. It was of course assumed in this exercise that the multimode method was "exact" i.e. that it give the correct answer against which results from the single mode method may be compared.

10.1.4 Input Data

Table 24 lists the SEISAB input data for the single mode analysis of the basic bridge. Even without any prior experience with SEISAB, it is possible to understand this input file. This is due, in large measure, to the development of a problem oriented language and the use of terms already familiar to a bridge engineer. Detailed explanations of each input command can be found in reference 41.

10.1.5 Results

Three sets of typical results are presented in Tables 25, 26 and 27 and follows:

- Table 25: Comparison of Periods of Vibration.
- Table 26: Comparison of Column Displacements.
- Table 27: Comparison of Column Forces and Moments.

These results have been extracted from the SEISAB output files and are used here to illustrate the range of output available and trends in response.

10.1.6 Discussion of Results

It appears from examination of table 25 that the single mode method gives similar periods of vibration to those predicted by the multimode method. This is clearly the case for the first four examples but it is not so clear for the second set of four examples. The reason for this is that, in the second set, the transverse release of the abutments permits a rotational mode to occur which is strongly coupled with the longitudinal and transverse modes. It is therefore not strictly correct to make period comparisons for these four examples because the single mode methods cannot model these coupled actions. In other words, although the periods can be compared for examples E through H in table 24, they are not for exactly the same mode of vibration.

The displacements in table 26 are for the second column in bent 2. Again good agreement is evident between the two methods until the transverse restraint is released at the abutments. As noted above, this is done in examples E through H and it introduces a rotational mode of vibration in each case. This mode is seen to be particularly excited during transverse loading. Consequently the transverse displacements

Table 24: SEISAB Input Data - Example 1

```
C ***************************************************************
C *                                                             *
C *                    E X A M P L E   1                        *
C *                                                             *
C *     SINGLE MODE ANALYSIS OF A STRAIGHT THREE-SPAN BRIDGE.   *
C *     BASE EXAMPLE - EQUAL COLUMNS, NO SKEW, ETC...           *
C *     UNITS USED ARE KIPS, FEET, SECONDS AND RADIANS          *
C *                                                             *
C ***************************************************************
C
SEISAB   'EXAMPLE 1, STRAIGHT THREE-SPAN BRIDGE'
  SINGLE MODE ANALYSIS
C
C -------- ALIGNMENT DATA BLOCK IS OMITTED ENTIRELY --------------
C
SPANS
  LENGTHS,120.75,120.5,134.167      $ SPAN LENGTHS MUST BE SPECIFIED
  AREA 123.0                        $ EXPLICITLY
  I11 117.0
  I22 65550.0                $ GENERATION OF SPAN CROSS SECTION DATA
  I33 527.0                  $ IS USED
  E 430000.0
  WEIGHT 0.125
DESCRIBE
  COLUMN 'TYPE 1' 'TYPICAL BENT COLUMN'
    AREA 13.0                $ NOTE: LENGTH INPUT IS NOT REQUIRED FOR
    I11,13.0                 $       SINGLE SEGMENT (PRISMATIC) COLUMN
    I22,13.0
    I33,13.0
    E 430000.0
  SPECIAL CAP 'TYPE A'    'SPECIAL CAP MEMBER IS USED FOR BENTS'
    I33  72.2
    E 440000.0
ABUTMENT STATION 0 + 0        $ VALUES FOR ABUTMENT 4 ARE IDENTICAL
  BEARING N 00 00 00 E        $ TO THOSE AT ABUTMENT 1 AND ARE
  ELEVATION,125.0             $ OBTAINED BY GENERATION
BENT
  BEARING N 00 00 00 E
  ELEVATION TOP 125.0
  HEIGHT 25.0
  COLUMN NORMAL LAYOUT 'TYPE 1' 35.0 'TYPE 1' 35.0 'TYPE 1' AT 2,3
  SPECIAL CAP 'TYPE A' AT 2,3
LOADS
  SINGLE MODE ANALYSIS
    ATC6 CURVE
      SOIL TYPE II
      ACCELERATION COEFFICIENT 0.4
FINISH
```

Table 25: Comparison of Transverse and Longitudinal Periods
Straight Bridge

EXAMPLE	PERIOD – seconds			
	SINGLE MODE		MULTI MODE (EIGEN)	
	Longitudinal	Transverse	Longitudinal	Transverse
A	0.610	0.312	0.624 (1) [1]	0.316 (4)
B	0.819	0.347	0.835 (1)	0.353 (4)
C	0.653	0.322	0.662 (1)	0.323 (4)
D	0.868	0.353	0.880 (1)	0.355 (4)
E	0.610	0.637	0.624 (3)	0.628 (2)
F	0.819	1.774	0.835 (2)	2.053 (1)
G	0.656	0.694	0.629 (3) [2]	0.722 (2) [2]
H	0.873	1.558	0.909 (2) [3]	1.827 (1) [3]

NOTES :

1. The numbers in parenthesis indicate mode numbers.
2. These modes (2 and 3) are strongly coupled.
3. These modes (1 and 2) are strongly coupled.

Table 26: Comparison of Absolute Column Displacements for Column 2 of Bent 2 – Straight Bridge

EXAMPLE	LOAD CASE	SINGLE MODE		MULTI MODE (EIGEN)	
		Longitudinal (feet)	Transverse (feet)	Longitudinal (feet)	Transverse (feet)
A	L	0.242	0.000	0.238	0.000
	T	0.000	0.085	0.000	0.087
	L+0.3T	–	–	0.238	0.026
	0.3L+T	–	–	0.071	0.087
B	L	0.357	0.000	0.361	0.000
	T	0.000	0.104	0.000	0.107
	L+0.3T	–	–	0.361	0.032
	0.3L+T	–	–	0.108	0.107
C	L	0.265	0.012	0.266	0.018
	T	0.012	0.091	0.013	0.090
	L+0.3T	–	–	0.270	0.044
	0.3L+T	–	–	0.093	0.095
D	L	0.386	0.012	0.393	0.016
	T	0.013	0.108	0.011	0.105
	L+0.3T	–	–	0.396	0.047
	0.3L+T	–	–	0.129	0.109
E	L	0.242	0.000	0.238	0.000
	T	0.000	0.155	0.000	0.254
	L+0.3T	–	–	0.238	0.076
	0.3L+T	–	–	0.071	0.254
F	L	0.357	0.000	0.361	0.000
	T	0.000	0.001	0.000	0.306
	L+0.3T	–	–	0.361	0.092
	0.3L+T	–	–	0.108	0.306
G	L	0.267	0.035	0.222	0.158
	T	0.033	0.209	0.151	0.245
	L+0.3T	–	–	0.267	0.232
	0.3L+T	–	–	0.218	0.293
H	L	0.389	0.052	0.356	0.203
	T	0.015	0.066	0.146	0.289
	L+0.3T	–	–	0.399	0.290
	0.3L+T	–	–	0.252	0.350

NOTE: SEISAB does not output the combination of the longitudinal (L) and transverse (T) analyses under the single mode option.

Table 27: Comparison of Maximum Column Forces and Moments for Bottom of Column 2, Bent 2 - Straight Bridge

EXAMPLE NO.	LOAD CASE	COLUMN FORCES AND MOMENTS			
		SINGLE MODE		MULTI MODE (EIGEN)	
		Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)
A	L	932.6	12,100	913.5	11,856
	T	358.7	4,518	373.1	4,655
	L+0.3T	–	–	913.5	11,856
	0.3L+T	–	–	373.1	4,655
B	L	1,337.1	17,525	1,327.7	17,516
	T	434.7	5,475	456.9	5,711
	L+0.3T	–	–	1,327.7	17,516
	0.3L+T	–	–	456.9	5,711
C	L	919.9	10,862	912.0	10,741
	T	206.3	3,024	220.2	3,000
	L+0.3T	–	–	978.0	11,641
	0.3L+T	–	–	493.8	6,223
D	L	1,333.3	15,838	1,345.6	15,987
	T	240.3	3,541	253.9	3,452
	L+0.3T	–	–	1,421.8	17,023
	0.3L+T	–	–	657.5	8,248
E	L	932.6	12,100	913.5	11,856
	T	651.2	8,204	1,071.9	13,469
	L+0.3T	–	–	913.5	11,856
	0.3L+T	–	–	1,071.0	13,469
F	L	1,337.1	17,525	1,327.8	17,516
	T	4.4	57	1,290.3	16,220
	L+0.3T	–	–	1,327.8	17,516
	0.3L+T	–	–	1,290.3	16,220
G	L	873.5	10,211	879.4	10,729
	T	487.1	7,120	662.3	9,683
	L+0.3T	–	–	1,072.7	13,224
	0.3L+T	–	–	908.1	12,111
H	L	1,250.3	14,689	1,044.2	14,733
	T	166.3	2,404	981.9	12,685
	L+0.3T	–	–	1,338.8	17,189
	0.3L+T	–	–	1,295.1	16,373

NOTE: The shears and moments listed are the maximum values and can be either longitudinal or transverse.

calculated by the single mode method are in substantial error for each of these four cases. However, it is also true that changes in column height and the introduction of skew can be accommodated quite successfully by the single mode method.

A similar trend is evident in table 27 which compares forces and moments at the bottom of the second column in bent 2. Once the abutments are released and the rotational mode is excited by transverse loading, the shear forces and moments are underestimated by the single mode method. In fact, in example F, these quantities bear no relation to the correct values as given by the multimode method. But it must be remembered that this is to be expected from a method which uses a single mode shape to represent (in this case) a bridge with at least two strongly coupled lateral modes of vibration. The method works well for bridges with single modes and, as shown here, gives good results even when column heights change dramatically from pier to pier and a large skew is present.

10.2 CURVED BRIDGE EXAMPLE

10.2.1 Geometry

A six span continuous girder bridge is supported on single columns as shown in figure 126. Properties for both the superstructure and substructure are given in this same figure. Of particular interest is the hinge near mid-length and the flexible soil conditions requiring explicit modelling of the foundation compliance. Restrainers are also provided at the abutments, columns and hinge. This example has been taken from the SEISAB "User Manual and Example Problems" [reference 41].

10.2.2 Load Cases

The response spectrum chosen to load this bridge was as follows.

Acceleration Coefficient, A = 0.4
Soil Profile Type: III, (S = 1.5)
Seismic load coefficient, $C_S = 1.2AS/T^{2/3}$
$= 0.720/T^{2/3}$
Directions: longitudinal (along the chord joining the two abutments; and transverse (normal to the chord)

Load combinations were made as follows:

Case 1: 1.0 * longitudinal + 0.3 * transverse
Case 2: 0.3 * longitudinal + 1.0 * transverse

10.2.3 Analysis Methods

Both single mode spectral analysis (procedure 1) and multimode spectral analysis (procedure 2) were used to analyze this bridge. As with the straight bridge, one purpose of this exercise was to determine the suitability of the single mode method for a bridge of this complexity.

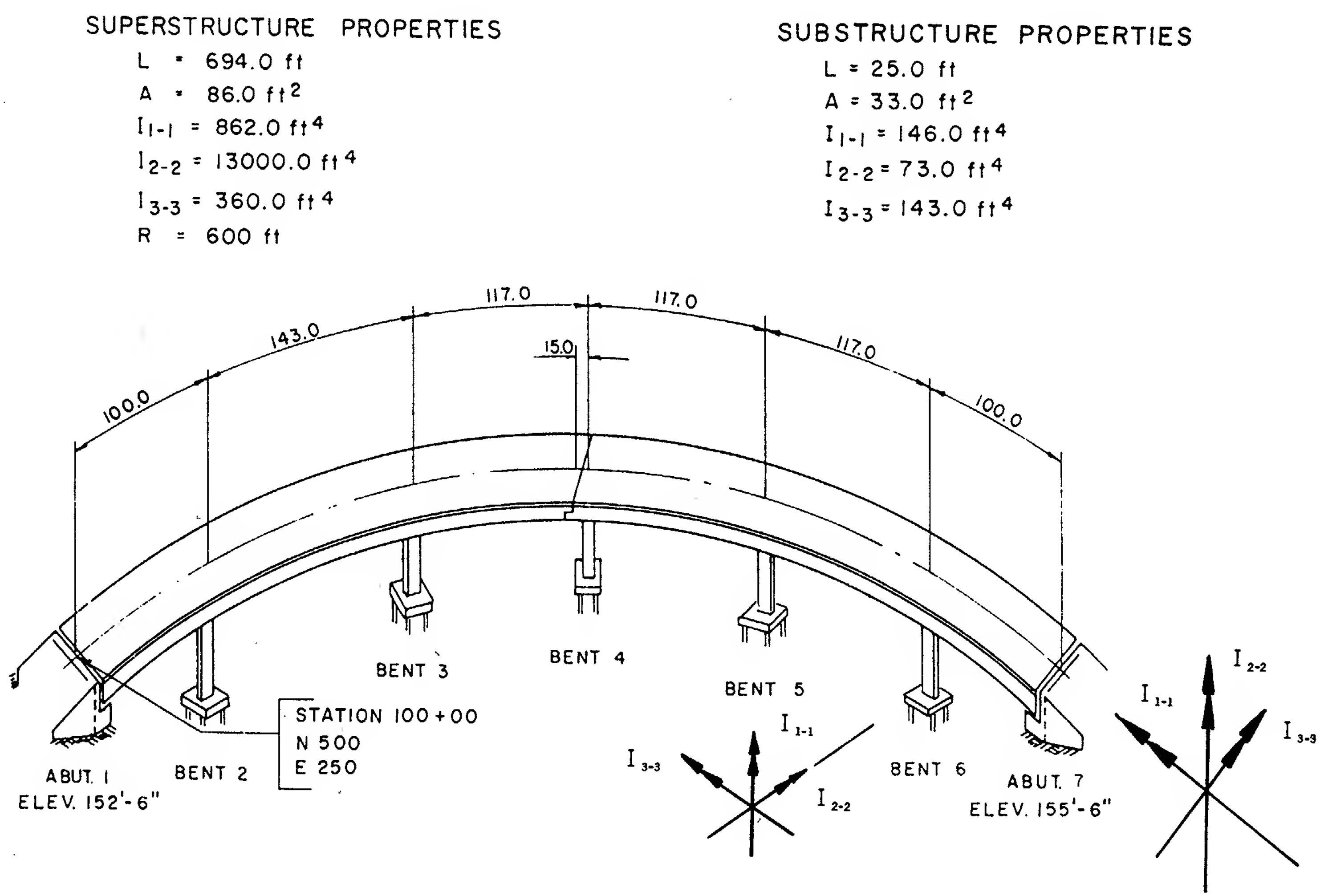


Figure 126: Six Span. Curved Bridge Example

Table 28: SEISAB Input Data - Example 2

```
C ****************************************************************
C *                                                              *
C *                    E X A M P L E   2                         *
C *                                                              *
C *     RESPONSE SPECTRUM ANALYSIS OF A SIX-SPAN CURVED BRIDGE   *
C *         UNITS USED ARE KIPS, FEET, SECONDS AND RADIANS       *
C *                                                              *
C ****************************************************************
C
SEISAB 'EXAMPLE 2: SIX-SPAN CURVED BRIDGE
  RESPONSE SPECTRUM ANALYSIS
  SUPERSTRUCTURE JOINTS 4      $ FOUR INTERMEDIATE SUPERSTRUCTURE
ALIGNMENT
  STATION 100 + 0.0
  COORDINATES    N 500.  E 250.
  BEARING        N 0 E
    BC 10000.0
    RADIUS    R 600.0
    BEARING   N 66,16,20,E
SPANS
  LENGTHS 100.0,143.0,3*117.0,100.0
  I33    360.0     $ GENERATION OF VALUES IS PERFORMED FOR SPANS 2-6
  I11    862.0     $ NOTE THAT COMMANDS MAY BE INPUT IN ANY ORDER
  I22 13000.0      $ FOLLOWING THE LENGTHS COMMAND
  AREA    86.0
C
C   DEFAULT VALUES FOR MATERIAL DENSITY AND ELASTIC MODULUS
C   ARE USED.
C
DESCRIBE
  COLUMN   'BNT COL' 'PRISMATIC BENT COLUMN TYPE'
    AREA 33.0
    I11 146.0
    I22 73.0            $ DEFAULT DENSITY AND MODULUS VALUES USED
    I33 143.0
  RESTRAINER 'TYPE 1' 'GALV.  H.S.  ROD'
    LENGTH    5.0
    AREA      3.068E-03
    E         2.01E+06
  RESTRAINER  'TYPE 2' 'GALV.  STEEL CABLE'
    LENGTH  20.0
    AREA    0.01                     $ DEFAULT MODULUS VALUE USED
ABUTMENT STATION  100 + 0.0
  ELEVATION            152.5 155.5
  WIDTH NORMAL         35.0          $ GENERATION FOR ABUT 7 IS USED
  RESTRAINER NORMAL LAYOUT 'TYPE 1' 8.0 8.0 'TYPE 1'           AT 1
  RESTRAINER NORMAL LAYOUT 'TYPE 1' 8.0 'TYPE 1' 8.0 'TYPE 1'  AT 7
BENT
  ELEVATION TOP  153.0,153.5,154.0,154.5,155.0
  HEIGHT 25.0          $ GENERATION OF HEIGHT DATA IS PERFORMED
  COLUMN   'BNT COL'  AT 2,3,4,5,6
```

Table 28: SEISAB Input Data – Example 2 (cont'd)

```
C                    EXAMPLE 2   Continued.........
C
C
FOUNDATION
  AT BENT  2,3,4,5,6
    SPRING CONSTANTS
      KM2M2   2.704E+10
      KM3M3   1.292E10        $ FOUNDATION DATA MAY BE INPUT IN
      KM1M1   2.2204E10       $ ANY ORDER FOLLOWING THE 'AT' COMMAND
      KF3F3   4.084E+08
      KF1F1   4.084E08
HINGE
  AT 3    102.00
    TRANSVERSE  PIN
    RESTRAINER NORMAL LAYOUT 'TYPE 2' 4.5,4.0,4.0,4.5 'TYPE 2' AT 1
    WIDTH NORMAL 33.5  AT 1
LOADS
  RESPONSE SPECTRUM
    ATC6 CURVE
      SOIL TYPE III
      ACCELERATION COEFFICIENT 0.4
C
C          THE DEFAULT DIRECTION FACTORS AND NUMBER OF MODES
C          ARE USED
C
FINISH
```

Table 29: Comparison of Transverse and Longitudinal Periods – Curved Bridge

EXAMPLE	PERIOD – seconds			
	SINGLE MODE		MULTI MODE (EIGEN)	
	Longitudinal	Transverse	Longitudinal	Transverse
Curved	0.340	0.365	0.380 (2) [1,2]	0.369 (3) [2]

NOTES :

1. The numbers in parenthesis indicate mode numbers.
2. These modes (2 and 3) are strongly coupled.

Table 30: Comparison of Absolute Column Displacements and Hinge Openings for Bent 4 – Curved Bridge

EXAMPLE	LOAD CASE	SINGLE MODE		MULTI MODE (EIGEN)	
		Longitudinal (feet)	Transverse (feet)	Longitudinal (feet)	Transverse (feet)
Curved	L	0.053	0.001	0.047	0.011
	T	0.016	0.109	0.009	0.116
	L+0.3T	–	–	0.050	0.046
	0.3L+T	–	–	0.023	0.119
Hinge Openings	L	0.041	0.000	0.067	0.000
	T	0.043	0.000	0.018	0.000
	L+0.3T	–	–	0.072	0.000
	0.3L+T	–	–	0.038	0.000

Table 31: Comparison of Maximum Column Forces and Moments for Bottom of Bent 4 - Curved Bridge

EXAMPLE NO.	LOAD CASE	COLUMN FORCES AND MOMENTS			
		SINGLE MODE		MULTI MODE (EIGEN)	
		Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)
Curved	L	902.4	12,873	790.9	11,180
	T	1,403.7	33,307	1,566.8	35,927
	L+0.3T	-	-	812.5	14,358
	0.3L+T	-	-	1,615.1	37,001

NOTE : The shears and moments listed are the maximum values and can be either longitudinal or transverse.

10.2.4 Input Data

Table 28 lists the SEISAB input data for the multimode analysis of this bridge. Again it is possible to read and understand this input file without prior knowldge of the SEISAB input commands, demonstrating once more, the usefulness of a problem-oriented input language. Details of each command and option are given in reference 41.

10.2.5 Results

Three sets of results are presented in tables 29, 30 and 31 as follows:

- Table 29 Comparison of Periods of Vibration.
- Table 30 Comparison of Column Displacements.
- Table 31 Comparison of Column Forces and Moments.

These results have been extracted from the SEISAB output file and are used here to illustrate the available output and the accuracy of the single mode method.

10.2.6 Discussion of Results

Because of the curved nature of the bridge, the longitudinal and transverse modes of the bridge are strongly coupled. The single mode method is therefore unable to represent this action and although the periods of vibration (table 29) are remarkably close, they are for different mode shapes. This inability to represent more than one mode at a time is evident in the displacement results given for the column of pier 4 by the single mode method (table 30). The transverse deflection during longitudinal loading is an order of magnitude too low and the longitudinal deflection during transverse loading is almost twice the correct value. However, the maximum shear forces and moments at the bottom of this column (table 31) are close to the multimode values, and this is generally true for the other columns in the bridge. It can therefore be concluded that the results for shear forces and moments from the single mode method are adequate for design purposes but that the displacements should be treated with caution.

APPENDIX A

MODIFIED MERCALLI INTENSITY SCALE
(1956 version)*

Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering.

Masonry A. Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

Masonry B. Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

Masonry C. Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D. Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

INTENSITY VALUE	DESCRIPTION
I.	Not felt. Marginal and long-period effects of large earthquakes.
II.	Felt by persons at rest, on upper floors, or favorably placed.
III.	Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.
IV.	Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV, wooden walls and frame creak.

---

* Original 1931 version in Wood, H.O. and Neumann, F., 1931, Modified Mercalli intensity scale of 1931: Seismological Society of America Bulletin, v. 53, no. 5, p. 979-987. 1956 version prepared by Charles F. Richter, in Elementary Seismology, 1958, pp.137-138. W.H. Freeman and Company.

V. Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.

VI. Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc. off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken visibly, or heard to rustle.

VII. Difficult to stand. Noticed by drivers. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices also unbraced parapets and architectural ornaments. Some cracks in masonry C. Waves on ponds, water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.

VIII. Steering of cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.

IX. General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.

X. Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

XI. Rails bent greatly. Underground pipelines completely out of service.

XII. Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

APPENDIX B: ABUTMENT DESIGN

As described in Chapter 5 and illustrated in figure 53, abutments can be divided into two classifications for seismic analysis:

1) Monolithic abutments (also known as integral, or end diaphragm abutments)
2) Seat type abutments (also known as free-standing abutments)

Monolithic abutments tend to mobilize the adjacent soils and can be very effective at absorbing energy during an earthquake, in both the longitudinal and transverse directions. When it is desirable to carry large forces into the soils, this abutment type is preferred. Damage may be heavy but with adequate reinforcement and a generous berm length, this abutment should perform satisfactorily and the collapse potential will be very low.

Seat type abutments permit the engineer more control over the degree of soil mobilization, but the added joint introduces a potential collapse mechanism into the structure. However, damage with this type of abutment will be less than with a monolithic abutment. Longitudinally, the backwall gap may be designed to permit larger or smaller movements to occur as desired. Transversely the superstructure may be held or released.

B.1 Abutment Stiffness

Abutments usually attract high in-plane forces during an earthquake because of their very high lateral stiffness. These structures are nearly rigid because of their physical configuration and the restraint provided by the approach fills. It is therefore important to include the combined stiffness effects of the abutment and backfill in any seismic analysis, particularly when calculating performance in the longitudinal direction.

However, the interaction effects between the soil and structure are complex and difficult to quantify numerically. Nevertheless it is important that some attempt be made to include these effects in a seismic analysis. The approximate procedure outlined in Section 7.4.2D is recommended as a minimum level of effort. This procedure is illustrated in the following sections, using examples taken from reference 38.

B.2 Initial Abutment Stiffness Calculation (Step 1, Section 7.2.4D)

The following preliminary stiffness coefficients are suggested (reference 38) for average abutment backfill conditions.

Wall/footing stiffness against backfill, K_s = 200 K/in per lineal foot of wall
Pile stiffness, K_p = 40 K/in per pile

K_s is based on material with shear velocity = 800 ft/sec and a wall height of about 8 ft. K_p is for OSD standard 45 ton, 70 ton and 16" CIDH piles.

All components which contribute to the abutment stiffness, in the direction under consideration, should be considered. Wingwalls should also be included but only if they can withstand the associated forces.

B.2.1 Longitudinal Stiffness

The full passive resistance of the soil is only mobilized when the abutment is moving towards the backfill. It is essentially zero when moving away from the backfill. Therefore only one equivalent abutment spring is effective at any one instant in time. If both abutments are identical, the equivalent bridge model will have just one abutment spring arbitrarily located at one or other end.

If the abutments are unequal, two trials may be necessary, with first one abutment spring and then the other to determine which produces the most severe set of forces and deflections.

The 50-foot wide abutment shown in figure 127a, is supported on 5 vertical piles, and comprises an 8 ft. high end wall and wingwalls which are 12 ft. long.

Using the stiffness coefficients of the previous section, the following equivalent abutment spring coefficient is calculated:

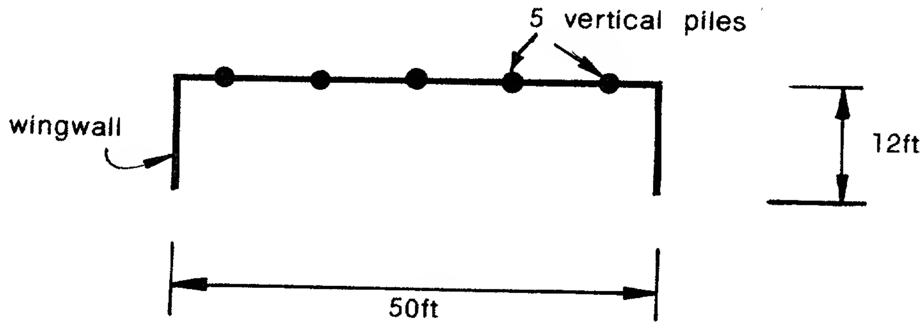
soil contribution = 200 K/in x 50 ft	=	10,000 K/in
pile contribution = 40 K/in x 10 piles	=	400 K/in
total abutment stiffness (initial estimate)	=	10,400 K/in (longitudinal at one abutment)

Note:
1. 10 piles are assumed effective, 5 at each abutment.
2. Only 1 endwall is assumed effective, at any point in time.
3. Wingwall contribution is neglected in longitudinal direction.

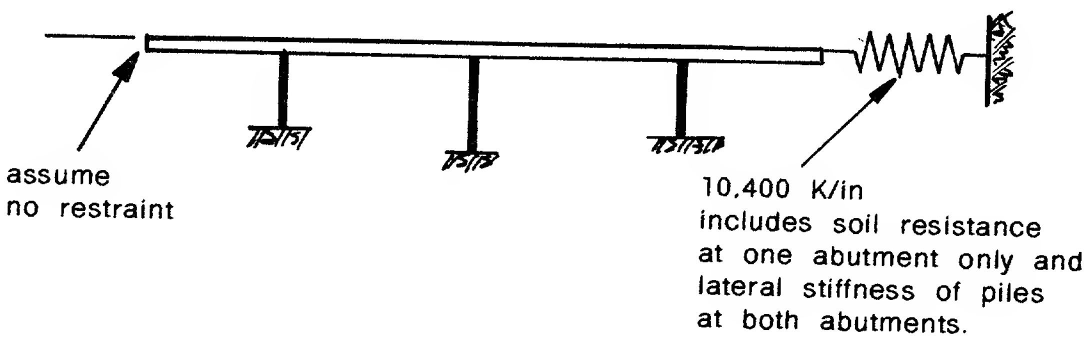
The equivalent spring model in the longitudinal direction is shown in figure 127b.

B.2.2 Transverse Stiffness

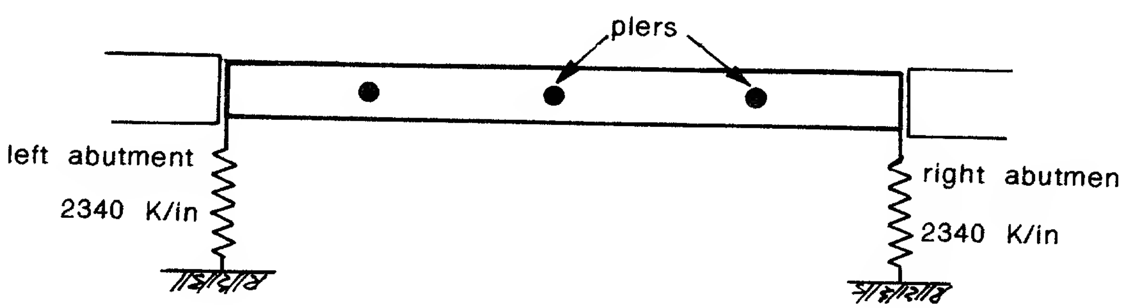
The soil resistance in the transverse direction is due to wingwall action only--the endwalls are assumed ineffective in this direction. However, because of the inherent flexibility in these walls, it is usual to downgrade their effective stiffness to, say, two-thirds of a fully restrained endwall. Also the two wingwalls at each abutment do not engage the same volume of backfill and a further reduction in stiffness is made to account for these variances. For example, the effective volume of soil that can be mobilized between the two walls is considerably greater than that outside either wall, and the number of effective walls should therefore be reduced from 2 to say 1-1/3. However, each abutment should be examined on its own merits and these assessments of wingwall performance should be reviewed using judgement and taking into acount local site conditions. With the above assumptions, the following equivalent abutment stiffnesses (in the transverse direction) may be calculated.



(a) Abutment Plan



(b) Bridge Elevation – Longitudinal Model



(c) Bridge Plan – Transverse Model

Figure 127: Preliminary Abutment Stiffness – Equivalent Spring Models

soil contribution = 200 K/in x 12 x 0.67 x 1.33 = 2140 K/in
pile contribution = 40 K/in x 5 = 200 K/in
total abutment stiffness (initial estimate) = 2340 K/in (transverse, at both abutments)

The equivalent spring model in transverse direction is shown in figure 127c.

B.3 Abutment Forces and Displacements (Steps 2 and 3, Section 7.2.4D)

Using the preliminary abutment stiffnesses of the previous section, the abutment forces and deflections may now be calculated. If these forces and deflections are excessive, the assumed stiffnesses will require adjustment and the analysis repeated.

As a guide, abutment forces are considered excessive if the corresponding effective stress in the soil behind the abutment exceeds 5 ksf (reference 38). Assuming the ratio of effective stress to maximum stress is 0.65, under cyclic loading, a maximum stress of 7.7 ksf may be attained before degradation in the soil stiffness begins. Higher allowable soil stresses might be allowed for spread footings in firm soil. Also for the purpose of this calculation, ultimate pile loads of 40 K per pile might be assumed for 45 ton and 70 ton piles.

Excessive deformation at one or both abutments is the most common form of seismic bridge damage. Past performance of bridge abutments has indicated that displacements more than 0.2 feet (2.4 in) cause severe problems, whereas movements less than this amount are repairable and cause little distress to the bridge structure. Accordingly, displacements more than 0.2 ft might be considered excessive for the purpose of these calculations.

B.3.1 Example: Abutment Design for Longitudinal Earthquake

The two span bridge in figure 128 has seat type abutments. The total deadload (W) is 10957 K. The seismic coefficient, A is 0.4g. The soil profile type is III and the site coefficient, S, is therefore 1.5. The preliminary abutment stiffness in the longitudinal direction is assumed zero because of the release provided by the bearings at the seats. This neglects the frictional forces inherent in sliders or the shear stiffness of elastomeric bearings if these are used at the seats. The initial value for longitudinal stiffness, K, is therefore governed by the pier alone and is shown to be 493 K/in.

period, $T = 2\pi\sqrt{\frac{W}{gK}} = 2\pi\sqrt{10957/384 \times 493} = 1.51$ secs.

seismic response coefficient $C_S = 1.2AS/T^{2/3}$
$= 1.2 \times 0.4 \times 1.5/(1.51)^{2/3}$
$= 0.55$

seismic force $= C_S.W = 0.55 \times 10957 = 6026$ K

deflection (longitudinal) = 6026/493 = 12.22 inches

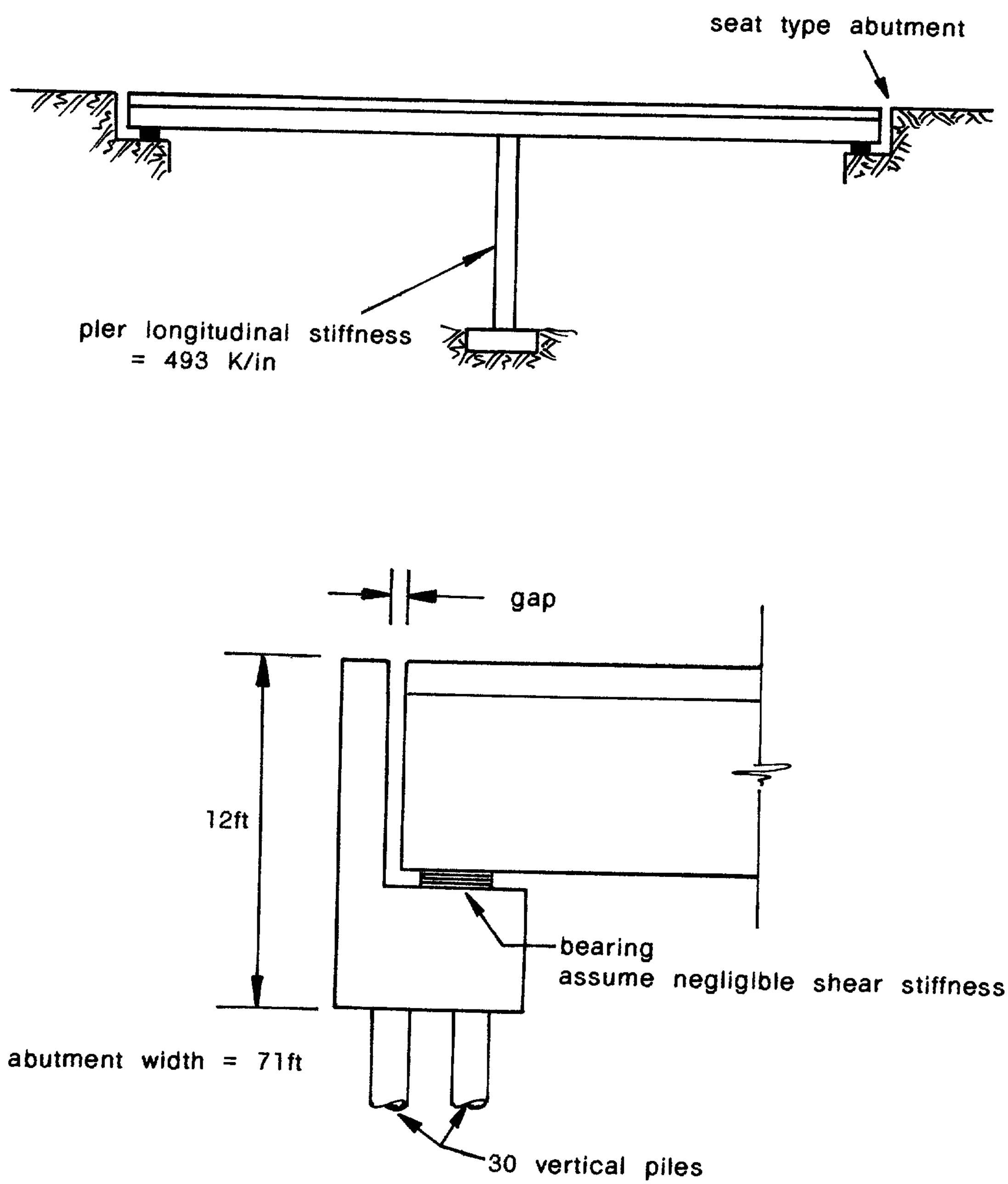


Figure 128: Example Abutment

Since abutment gaps are usually less than 6 inches, impact on the backwall will occur. The longitudinal stiffness of the abutment and backfill, which was neglected in the above calculation, must be therefore included in the analysis.

First calculate the abutment longitudinal stiffness.

Given: abutment width = 71 feet
number of vertical piles = 30
soil stiffness = 200 K/in/ft length of wall
pile stiffness = 40 K/in/pile

the abutment stiffness = 71 x 200 + 30 x 40
= 15,400 K/in

Also calculate the maximum abutment force that can be resisted before the soil reaches its limit state (maximum stress = 7.7 ksf) and the piles reach their ultimate load (40 K each).

backwall contribution = 7.7 x 12 x 71 = 6560 K
pile contribution = 30 x 40 = 1200 K
total (maximum abutment force) = 7760 K
= 7800 K (say)

Several trial analyses may now be necessary because of the nonlinear nature of the abutment force-deflection curve.

TRIAL 1. Assume total movement of superstructure will be 6 inches and specify the gap to be 4 inches. This implies 2 inches of abutment movement, which is less than the 2.4 inches previously given as a reasonable upper limit.

Figure 129 shows the result of this trial analysis. It is constructed as follows:

a. The first 4 inches of displacement occur "freely" and the structure stiffness (493 K/in) governs.

b. Once the 4 inch gap is closed, the combined stiffness of the structure and abutment (493 + 15,400 ≐ 15,900 K/in) governs. After about one half inch (7800/15,900) of deflection, the abutment strength (7800 K) is exceeded.

c. The remaining 1-1/2 inches (to a total of 6 inches) of deflection occurs after soil and pile failure and since it is assumed that there is now no stiffness contribution from the soil or piles, the structure stiffness (493 K/in) governs.

d. Calculate the total force at 6 inches of displacement as follows:
493 x 4 + 7800 + 493 x 1.51 = 10,516 K

e. Calculate the effective combined abutment and structure stiffness as follows:
10516/6 = 1753 K/in

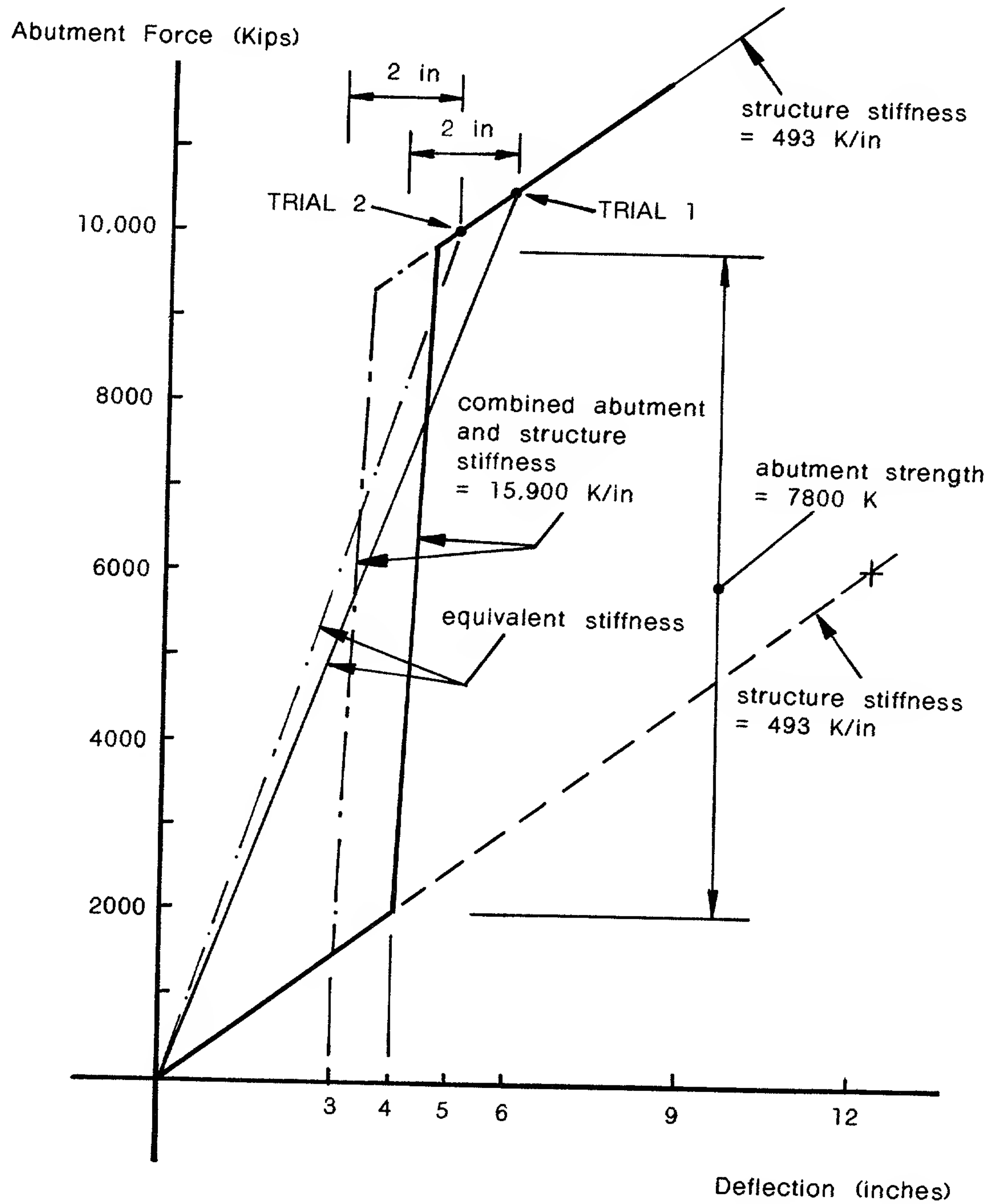


Figure 129: Abutment Trial Designs

Given this revised stiffness (K_1) recalculate the period of vibration,

$$T = 2\pi \sqrt{\frac{W}{gK_1}}$$

$$= 2\pi \sqrt{10957/384 \times 1753} = 0.80 \text{ sec.}$$

Then the revised seismic coefficient (C_s) $= 1.2 \times 0.4 \times 1.5/0.8^{2/3}$
$= 0.84$

and the seismic force = 0.84 x 10957 = 9200 K.

It follows that the longitudinal deflection = 9200/1753 = 5.2 inches

Since this is less than the 6.0 inches assumed for this trial, the 4-inch gap will be satisfactory. The abutment movement will be less than 2.4 inches. To calculate the actual deflection, a reanalysis must be made assuming 5 inches of displacement and a revised effective stiffness. However, a more economic design can usually be achieved by reducing the size of the gap, and for this example, the next trial assumes only a 3 inch gap and 5 inches of total superstructure movement.

TRIAL 2 Assume the total movement of the superstructure will be 5 inches, and the gap to be 3 inches. Figure 129 shows the results of this trial, which follows the steps used in Trial 1.

It can be seen that the total force at 5 inches of displacement is

493 x 3 + 7800 + 493 x 1.51 = 10,023 K

The effective stiffness is now 10,023/5 = 2005 K/in
The corresponding period of vibration = 0.75 secs
The seismic response coefficient = 0.87
and the corresponding seismic force = 9533 K
It follows that the deflection = 9642/2005 = 4.8 ins
which is satisfactorily close to the assumed value of 5 ins.

The gap can therefore be 3 inches wide, and the expected abutment movement will then be of the order of 1.8 inches which is less than the 2.4 inch upper limit recommended for damage control.

B3.2 General

Generally, one of four options may be selected in the longitudinal design of an abutment:

1. Provide a very large gap in order to isolate the superstructure movements from the abutment. This implies the use of a seat type abutment and, in the above example, would mean a gap in excess of 12 inches.

2. Provide a gap for thermal considerations and some seismic displacement but also expect abutment movement under seismic conditions. Design the abutment so that this movement is less than 0.2 feet.

3. Permit the abutment backwall to fail before exceeding the 0.2 foot limitation, thus protecting the abutment footing and pile unit. To be effective, the backwall (or a section of this wall) must be specifically detailed as a sacrificial unit, so as to fail at a predetermined level of load.

4. Permit the total abutment to move more than 0.2 feet.

Usually the fully free condition (1) is more expensive because of the larger road joint. However, with stiff piers, a small gap (which may also be necessary for temperature) may be adequate for seismic conditions.

Options (2) and (3) are about equal and the selection will depend on abutment size, gap requirements, soil type and whether or not backwall failure or part thereof, can be tolerated.

Option (4) will allow more movement and damage to occur at the abutment and will require an evaluation of stability of the total bridge. The use of Option (4) is quite valid for lower seismic areas and for bridges with adequate stability to survive the effects of large abutment movements.

GLOSSARY

Abutment
The substructure supporting the end span of a bridge at the approach to that span. Abutments may be of wall type or the spill-through type founded on piles or footings. They may be monolithic with the superstructure or structurally separate. In the former case, the backwall is also an end diaphragm for the bridge superstructure and hence the alternate name of end diaphragm abutment. A third alternate name is integral abutment. In the latter case, bearings to support the superstructure are necessary and a road joint is also required. These abutments are called seat type abutments, and sometimes free standing abutments, because the abutment can stand alone from the superstructure.

Accelerogram
The record from an accelerograph showing acceleration as a function of time.

Accelerograph
A strong motion earthquake instrument which records acceleration.

Aftershock
An earthquake, usually one of a series which may occur soon after the occurrence of a large earthquake (main shock), but with a magnitude smaller than the main event.

Amplification
An increase in earthquake motion within a structure because of the characteristics of the structure.

Amplitude
Maximum deviation of any time-varying quantity from its mean value during one half cycle of vibration.

Aseismic Region
One that is relatively free of earthquakes.

Attenuation
Reduction of earthquake intensity due to energy dissipation over distance with time.

Base Isolation
A technique for reducing earthquake motions and forces in a structure by isolating it from the ground at its base using flexible mounts with or without mechanical energy dissipators.

Base Shear
Total shear force acting at the base of a structure.

Bearings
Mechanical devices designed to permit relative movements between a structure and its foundation (or between parts of the same structure) while at the same time transmitting gravity loads. Used frequently in bridges and in all base-isolated structures.

Bent
Two or more columns in the same plane and joined by a capping beam. Commonly used for bridge substructures.

Bilinear
Representation by two straight lines of the stress versus strain properties of a material, one straight line to the yield point and a second line to represent post-elastic behavior.

Braced Frame
A framework of columns and beams that is stiffened (or braced) by additional members, usually diagonal ties.

Brittle Failure
Failure in a material without plastic deformation; such a failure occurs suddenly and usually without warning.

Connection
The structural detail which joins separate members together and which transfers one or more actions (forces and/or moments) from member to member.

Critical Damping
The minimum damping that will allow a displaced system to return to its initial position without oscillation.

Damping
The reduction in amplitude of vibration by energy absorption.

Diaphragm
A structural member which is intended to be rigid in its own plane and used to distribute external loads among two or more resisting members. In a building, floor slabs act as diaphragms for distributing seismic loads to columns and walls. In a bridge, the deck slab and transverse girders will act as a diaphragm, distributing lateral seismic loads to piers and/or abutments.

Ductility
Ability to withstand inelastic strain without fracturing due to plastic deformation.

Dynamic
Having to do with bodies in motion.

Effective Peak Acceleration
A normalizing factor for the construction of smoothed elastic acceleration response spectra (ATC-3-06 ground motion parameter).

Elasticity
The ability of a material to return immediately to its original form or condition after a displacing force is removed.

Elastomeric Bearing
A bearing constructed from a reinforced elastomer so that it is flexible (soft) in one plane and almost rigid normal to this plane. Rubber pads reinforced with thin steel plates are commonly used for bridge bearings.

Energy Dissipation (Absorption)
The loss of energy (usually as heat) during the inelastic deformations of structural materials. Energy dissipated in this way effectively dampens vibratory motions. Tolerates high plastic deformations without fracture (i.e., are ductile).

Epicenter
The point on the earth's surface vertically above the focus or hypocenter of an earthquake.

Failure Mode
The manner in which a structure fails (e.g., column buckling, shear failure, overturning of structure).

Fault
Planar or gently curved fracture in the earth's crust across which relative displacement has occurred.

Fault Zones
A fault zone consists of numerous interlacing small faults and may be many thousands of feet wide.

Felt Area
Total extent of area where the same earthquake is felt.

Flexible System
A system that will sustain relatively large displacements without failure.

Focal Depth
Depth of the earthquake focus (or hypocenter) below the ground surface.

Focus
Focus of an earthquake is the point at which rupture occurs; synonymous with hypocenter. (It marks the origin of the elastic waves of an earthquake.)

Frequency
The number of events which occur in a unit of time, usually measured in cycles per second.

Fundamental Period
The longest period (duration in time of one full cycle of oscillatory motion) of vibration of a structure which has several possible modes of vibration, each with a different period.

Ground Failure
Collapse of the ground due to landsliding, mud flows, liquefaction or similar catastrophe.

Free-Standing Abutment
An abutment which is structurally separate from the superstructure.

Ground Movement
A general term; includes all aspects of motion (acceleration, particle velocity, displacement).

Ground Acceleration
Acceleration of the ground due to earthquake forces.

Ground Velocity
Velocity of the ground during an earthquake.

Ground Displacement
The distance which the ground moves from its original position during an earthquake.

Hold-Down Devices
Mechanical devices for restraining simply supported bridge spans from falling off pier and abutment seats during an earthquake.

Hypocenter
The point below the epicenter at which an earthquake actually begins; the focus.

Importance Classification
An assessment of the importance of structure which is then used to determine the appropriate level of seismic design loads. For bridges, two classifications are used: essential and nonessential.

Inelastic Behavior
Behavior of an element beyond its elastic limit.

Intensity
A subjective measure of the force of an earthquake at a particular place as determined by its effects on persons, structures and earth materials. Intensity is a measure of effects as contrasted with magnitude which is a measure of energy. The principal intensity scale used in the United States today is the Modified Mercalli, 1956 version.

Irregular Bridge
A bridge that does not satisfy the definition of a regular bridge (See **Regular Bridge**.

Lateral Force Coefficients
Factors applied to the weight of a structure or its parts to determine lateral force for aseismic structural design.

Linkage
A mechanical device (usually a bolt) which may be used to tie several parts of a bridge superstructure together or to an abutment wall for the same purpose as a hold-down device.

Liquefaction
Transformation of a granular material (soil) from a solid state into a liquefied state as a consequence of increased pore-water pressure induced by vibrations.

Lumped Mass
An assumption made to simplify dynamic analysis in which actual distributions of mass are lumped together at specific locations based on tributary volumes of material.

Macrozones
Large zones of earthquake activity such as zones designated by the ATC-6 Acceleration Coefficient map.

Magnification Factor
An increase in the induced lateral forces in a structure due to frequency matching between the ground and structure.

Magnitude
A measure of earthquake size which describes the amount of energy released. See **Richter Magnitude Scale**.

Mantle
The main bulk of the earth between the crust and core, varying in depth from 40 to 3,480 kilometers.

Mean Return Period
The average time between occurrences of an earthquake of a given size at a particular location.

Modal Analysis
Determination of design earthquake forces based upon the theoretical response of a structure in its several modes of vibration.

Modified Mercalli Scale
See **Intensity**.

Mode Shape
The characteristic shape of a vibrating structure. Complex structures can vibrate in more than one mode shape, depending on the exciting frequency, and respond to earthquakes in a combination of mode shapes.

Moment Frame
One which is capable of resisting bending moments at the joints, enabling it to resist lateral forces or unsymmetrical vertical loads through overall bending action rather than bracing. (See **Braced Frame**).

Monolithic Abutment
An abutment which is integral with the superstructure and may not have a road joint.

Multimode
More than one mode of vibration. Vibration in a complex structure (e.g., a long bridge) usually consists of a combination of several modes and is said to be multimodal.

Natural Frequency
The frequency of free vibration of a structure if damping effects are neglected. Sometimes expressed in radians/sec.

Nonstructural Components
Those components which are not intended for the structural support of a bridge; e.g., handrail.

Normalization
A method of standardizing vibration characteristics.

Out of Phase
A vibration state where a structure in motion is not moving in the same direction as the ground or where different parts of the same structure are not moving together. Long bridges exhibit this behavior when different pier footings and abutments are moving in opposing directions.

Period
The time for an oscillatory event to occur and usually measured in seconds; the inverse of frequency. It is the amount of time taken for an oscillating system to complete one cycle of motion. It is analagous to the time which elapses between successive high points of the swing of a pendulum.

Pier
Any support to a bridge superstructure which is located between the two abutments.

Pile
A column which is either driven or cast in the ground to support the weight of a structure.

Pile Cap
A beam or slab which joins two or more piles together at the ground surface. It is also used as the foundation beam from which the pier or abutment is erected.

Plate Tectonics
The theory and study of plate formation, movement, interaction, and destruction; the theory which explains seismicity, volcanism, mountain building and paleomagnetic evidence in terms of plate motions.

Plastic Hinge
A hinge in a beam or column which is formed when the material at that point reaches its fully yielded state and rotation occurs without a significant increase in load. Formation of a sufficient number of plastic hinges in a structure can lead to a collapse mechanism.

Regular Bridge
A bridge that has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports (abutments excluded).

Resonance
A state of maximum amplitude of vibration caused by the exact matching of the excitation frequency with the natural frequency of the structure itself.

Response
Behavior of a structure excited by earthquake ground motion and measured by structure displacement or member action.

Response Modification Factors
These factors are used to modify the component forces calculated from an elastic analysis of a structure to determine design forces. These factors are based on the assumption that columns will yield when subjected to forces induced by actual seismic loads and that connections and foundations must be designed to resist these same loads with little or no damage.

Response Spectrum
See **Spectra.**

Retaining Wall
A wall to support the ground from slipping; frequently used at an abutment to retain the approach fill.

Richter Magnitude Scale
A measure of earthquake size which describes the amount of energy released. The measure is determined by taking the common logarithm (base 10) of the largest ground motion observed during the arrival of a P-wave or seismic surface wave and applying a standard correction for distance to the epicenter.

Rigidity
Relative stiffness of a structure or element. In numerical terms, equal to the force required to cause a unit displacement.

Seismic
Pertaining to earthquake activities.

Seismicity
The world-wide or local distribution of earthquakes in space and time; a general term for the number of earthquakes in a unit of time, or for relative earthquake activity.

Seismic Performance Category
A classification system which reflects the importance of a bridge and the variation in seismic risk throughout the country. The ATC-6 "Seismic Design Guidelines for Highway Bridges" defines four categories to permit variations in the methods of analysis, minimum support lengths, column design details, and foundation and abutment design requirements in accordance with the seismic risk associated with a particular bridge location.

Shear Distribution
Distribution of lateral forces along the height or width of a building or between the piers and abutments in a bridge.

Shear Strength
The stress at which a material fails in shear.

Shear Wall
A wall designed to resist lateral forces parallel to the wall. A shear wall is normally vertical, although not necessarily so.

Shear Key
A structural component intended to transfer shear force from one structural member to another.

Simple Harmonic Motion
Oscillatory motion of an object at a single frequency. Essentially a vibratory displacement such as that described by a weight which is attached to one end of a spring and allowed to vibrate freely. This displacement may be described by a sinusoidal function of time.

Single-Mode
Vibration of a structure is completely described by one mode of vibration rather than a combination of modes as in complex structures. See **Multimode**.

Site Effects
The effect of site soil conditions on the behavior of a bridge or building during an earthquake.

Soil-Structure Interaction
See **Site Effects**.

Spectra
A plot indicating maximum earthquake response with respect to natural period or frequency of the structure or element. Spectra can show acceleration, velocity, displacement, shear or other measure of response.

Stability
Resistance to displacement or overturning.

Stiffness
Rigidity, or the reciprocal of flexibility.

Strain Release
Movement along a fault plane; can be gradual or abrupt.

Substructure (of a Bridge)
Those structural units which support the superstructure and therefore may include the pier cap, pier, pile cap, footing, piles, abutment and wingwall structures.

Superstructure (of a Bridge)
That part of a bridge which supports the live load and transfers this load to the substructures. It may comprise beams, slabs, box girders, and/or trusses.

Support Length
The length available at a pier or abutment to support the superstructure.

Time Dependent Response Analysis
Study of the behavior of a structure to determine a complete record or time history of response to input excitation.

Torsion
Twisting around an axis.

Vibration
A periodic motion which repeats itself after a definite interval of time.

Yield
The onset of plastic deformation in a member once the elastic limit has been exceeded. Accompanied by nonrecoverable deformations.

REFERENCES

[1] -----, "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials (AASHTO), 13th Edition, 1983 and Interim Specifictions 1984 and 1985.

[2] -----, "Standard Specifications for Highway Bridges Relating to Seismic Design," California Department of Transportation, Division of Structures, 1985.

[3] -----, "Seismic Design Guidelines for Highway Bridges," Report ATC-6, Applied Technology Council, 1981. Also published by Federal Highway Administration as Report FHWA/RD-81/081 1981.

[4] -----, "Guide Specifications for Seismic Design of Highway Bridges," American Association of State Highway and Transportation Officials (AASHTO) 1983.

[5] -----, "Seismic Retrofitting Guidelines for Highway Bridges," Report ATC-6-2, Applied Technology Council, 1983. Also published by Federal Highway Administration as Report FHWA/RD-83/007, 1983.

[6] -----, "Seismic Design of Highway Bridge Foundations," Federal Highway Administration, Report in 3 Volumes: FHWA/RD-86/101, FHWA/RD-86/102, and FHWA/RD-86/103, June 1986.

[7] Bolt, B. A., "Earthquakes: A Primer", W. H. Freeman and Company, 1978.

[8] Dowrick, D. J., "Earthquake Resistant Design: A Manual for Engineers and Architects", John Wiley and Sons, 1977.

[9] Housner, G. W. and Jennings, P. C., "Earthquake Design Criteria", Earthquake Engineering Research Institute Monograph Series, 1982.

[10] Richter, C. F., "Elementary Seismology", W. H. Freeman and Company, 1958.

[11] -----, "Earthquake Engineering", Weigel, R. L., Coordinating Editor, Prentice-Hall, 1970.

[12] Gutenberg, B., and Richter, C. F., "Seismicity of the Earth," Princeton University Press, 2nd ed., 1954.

[13] Donovan, N. C., "A Statistical Evaluation of Strong Motion Data Including the February 9, 1971 San Fernando Earthquake", Proceedings, Fifth World Conference on Earthquake Engineering, Rome, 1973.

[14] Esteva, L., "Geology and Predictability in the Assessment of Seismic Risk", Proceedings, Second International Conference of the Association of Engineering Geologists, Sao Paulo, 1974.

[15] Newmark, N. M. and Rosenblueth, E., "Fundamentals of Earthquake Engineering", Prentice-Hall, 1971.

[16] Seed, H. B., and Idriss, I. M., "Ground Motions and Soil Liquefaction," Earthquake Engineering Research Institute Monograph, 1982.

[17] Clough, R.W. and Penzien, J., "Dynamics of Structures," McGraw-Hill Book Company, 1975.

[18] Biggs, John M., "Introduction to Structural Dynamics," McGraw Hill, 1964.

[19] Chopra, A., "Dynamics of Structures: A Primer," Earthquake Engineering Research Institute Monograph, 1981.

[20] Park, R. and Paulay, T., "Reinforced Concrete Structures," John Wiley & Sons, New York, 1975.

[21] Iwasaki, T., Penzien, J., and Clough, R.W., "Literature Survey - Seismic Effects on Highway Bridges," EERC Report No. 72-11, University of California, Berkeley, 1972, and FHWA-RD-73-13, November 1973.

[22] Cooper, J.D., "Bridge and Highway Damage Resulting from the 1976 Guatemala Earthquake," FHWA-RD-76-148, Office of Research and Development, Federal Highway Administration, Washington, D.C., May 1976.

[23] Imbsen, R.A., "Highway Structure Damage Caused by the Trinidad-Offshore, California Earthquake of November 8, 1980," FHWA, Washington, D.C., Report No. FHWA/RD-82/017, December 1981.

[24] Iwasaki, T., "A Case History of Bridge Performance during Earthquakes in Japan," Proc. Intl. Conf. on Case Histories in Geotechnical Engineering, St. Louis, May 1984. Also published as Technical Memorandum No. 2046, Public Works Research Institute, Ministry of Construction, Tsukuba Science City, Japan, February 1984.

[25] -----, "Highway Bridge Design Brief," Publication CDP 701/D (1978) and Amendments (1985), New Zealand Ministry of Works and Development, Wellington, 1985.

[26] -----, "Specifications of Highway Bridges and Commentaries; Part V: Earthquake-Resistant Design," Japan Road Association, May 1982.

[27] Priestley, M.J.N. and Park, R., "Strength and Ductility of Bridge Substructures,". Bulletin 71, Road Research Unit, Wellington, 1984.

[28] -----, "Manual of Steel Construction," American Institute of Steel Construction, Part 2, 8th Edition, Chicago, 1980.

[29] -----, "Seismic Design of Bridges," Bulletin 56, Road Research Unit, NZ National Roads Board, 1981, pp.94. Also published in Bulletin NZ National Society of Earthquake Engineering, Vol. 13, No. 3, 1980, pp.226-309.

[30] -----, "End Diaphragm Abutments," Caltrans Memo to Designers, 5-2, 1983.

[31] -----, "Roadway Drainage at Overcrossings on Depressed Freeways," Caltrans Memo to Designers, 18-4, 1975.

[32] Elms, D.G. and Martin, G.R., "Factors involved in the seismic design of bridge abutments," Proc. Workshop Earthquake Resistance of Highway Bridges, Applied Technology Council Project ATC-6-1, 1979.

[33] Wilson, E.L., Der Kiureghian, A. and Bayo, E.P., "A replacement for the SRSS method in seismic analysis," Journal Earthquake Engineering and Structural Dynamics, Vol. 9, 187-194, 1981.

[34] Gates, J. H., and Smith, M. J., "Verification of Dynamic Modeling Methods by Prototype Excitation," Federal Highway Administration Report FHWA/CA/SD-82-07, November 1982.

[35] -----, "Bridge Design Aids - Pile Shaft Design Procedure," California Department of Transportation, Division of Structures, August 1985.

[36] Davisson, M.T., "Lateral load capacity of piles," Highway Research Record No. 333 - Pile Foundations; Highway Research Board, 1970.

[37] Douglas B., Saiidi M., Richardson J., and Hart J., "Results from high amplitude dynamic tests and implications for seismic design," Seismic Reseach for Highway Bridges (U.S.-Japan Program) compiled by John F. Fleming, University of Pittsburgh, 1984.

[38] -----, "Earthquake Design Criteria - Commentary," California Department of Transportation, Division of Structures, Memo 15-10, July 1984.

[39] Bathe, K.J., Wilson, E.L., and Peterson, F.E., "SAP IV: A Structural Analysis Program for Static and Dynamic Response of Linear Systems," Report No. EERC-73/11, Earthquake Engineering Research Center, University of California, Berkeley, 1974.

[40] Logcher, R.D., Flachsbart, B.B., Hall, E.J., Power, C.M. and Ferrante, A.J., "ICES STRUDL II The Structural Design Language, Engineering User's Manual, Volume 1, Frame Analysis," MIT Department of Civil Engineering Report R68-91, November 1968. Proprietary versions also available as STRUDL, STRUDL DYNAL and STRUDL PLOTS from MCAUTO and Multisystems, Inc.

[41] Imbsen, R.A., Lea, J., Kaliakin, V., Perano, K., Gartes, J., and Perano, S., "SEISAB-I - SEISmic Analysis of Bridges: User Manual and Example Problems," Enginering Computer Corporation, Sacramento, California, January 1984.

HRT-10/8-87(700)QE
HHI-22/8-87(400)QE